ERDC/GSL SR-15-2

Geotechnical and Structures Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

ERDC
INNOVATIVE SOLUTIONS for a safer, better world

State-of-Practice on the Dynamic Response of Structures Strengthened with Fiber Reinforced Polymers (FRPs)

Esteban Pabon, Yomar Otero, Felipe J. Acosta,
Robert D. Moser, C. Kennan Crane, and Brian H. Green

July 2015

ERDC/GSL SR-15-2
July 2015

State-of-Practice on the Dynamic Response of Structures Strengthened with Fiber Reinforced Polymers (FRPs)

Robert D. Moser, C. Kennan Crane, Brian H. Green

Geotechnical and Structures Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Rd.
Vicksburg, MS 39180

Esteban I. Pabón, Yomar Y. Otero, and Felipe J. Acosta

Department of Civil Engineering and Surveying
University of Puerto Rico at Mayaguez
Call Box 9000
Mayaguez, PR 00681

Final report

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Abstract

Fiber Reinforced Polymers (FRPs) are one the most innovative materials for strengthening and retrofitting structures and structural elements due to their high modulus of elasticity, light weight, and other valuable properties. FRPs are often used in reinforced concrete, since concrete has a low modulus of elasticity and is more susceptible to exhibit cracks and brittle behavior. However, FRPs are not frequently used to repair steel structures because they are more ductile and have a relative high modulus of elasticity compared to concrete. In this work, a literature review for retrofitting structures using FRPs is presented. It is divided into three principal categories, concrete structures, steel structures, and masonry structures. The review shows that there is a vast collection of information on strengthening and retrofitting concrete elements with FRPs. The results show that using FRPs in certain concrete elements provides increasing ductility and shear capacity of a structure leading to a favorable seismic behavior. There is also valuable information for strengthening concrete structures that are susceptible to blast loads. When compared to reinforced concrete, the applications and available research on strengthening steel elements with FRPs is limited. Specific FRP applications that are being developed are discussed in this report.

Contents

Figures

Figures

Preface

This study was conducted for the Geotechnical and Structures Laboratory, U.S. Army Engineer Research and Development Center. The technical monitor was Dr. Robert D. Moser.

The work was performed by the University of Puerto Rico at Mayaguez (UPRM) and the Concrete and Materials Branch (CMB) of the Engineering Systems and Materials Division (ESMD), and the Structural Engineering Branch (StEB) of the Geosciences and Structures Division (GSD), U.S. Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Christopher M. Moore was Chief, CMB; Charles W. Ertle was Chief, StEB; Dr. Larry N. Lynch was Chief, ESMD; Bartley P. Durst was Chief, GSD, and Pamela G. Kinnebrew was the Technical Director for Military Engineering. The Acting Deputy Director of ERDC-GSL was Dr. Gordon W. McMahon, and the Acting Director was Dr. William P. Grogan.

LTC John T. Tucker III was the Acting Commander of ERDC, and Dr. Jeffery P. Holland was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
foot-pounds force	1.355818	joules
inches	0.0254	meters
inch-pounds (force)	0.1129848	newton meters
microinches	0.0254	micrometers
microns	1.0 E-06	meters
mils	0.0254	millimeters
pounds (force)	4.448222	Newtons
pounds (force) per foot	14.59390	newtons per meter
pounds (force) per inch	175.1268	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (mass) per cubic inch	2.757990 E+04	kilograms per cubic meter
pounds (mass) per square foot	4.882428	kilograms per square meter
pounds (mass) per square yard	0.542492	kilograms per square meter
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
tons (force)	8,896.443	newtons
tons (force) per square foot	95.76052	kilopascals
yards	0.9144	meters

1 Introduction

Fiber Reinforced Polymers (FRPs) are one of the most innovative materials in the engineering field today. Its high modulus of elasticity, low weight-to-strength ratio, versatility, and other characteristics make it a highly attractive material for use in the aerospace industry. In structural engineering, FRPs can solve problems like insufficient tensile capacity of reinforced concrete sections or provide adequate confinement for concrete. Application of FRP to strengthen concrete structures began during the 1980s (Bakis et al. 2002). Experimental research has mainly focused on the static and pseudo dynamic response of structural members and connections. However, the dynamic response of structures retrofitted with FRP has been less frequently studied due to the complexity of the response, e.g., the dynamic responses of structures subjected to earthquakes and blast loads. The American Concrete Institute (ACI) committee 440 is leading efforts to create design guidelines for the reinforcement and strengthening of concrete structures with FRPs. Current guidelines do not incorporate seismic or blast design, but there is an ongoing effort to incorporate these effects in the design guidelines. Blast loading effects are considered now under ACI committee 370 Blast and Impact Load Effects by a document in progress entitled “Design guidelines for blast strengthening of concrete and masonry structures using Fiber-Reinforced Polymer (FRP).” Seismic provision guidelines are under development to be incorporated in the ACI 440.2R document (ACI 2008).

FRPs used to retrofit concrete structures improve structural performance during earthquakes. For example, in previous earthquake events it was reported that several existing reinforced concrete (RC) frame structures that were designed according to pre-1970 codes experienced severe damage or even collapsed. This was mainly due to the fact that pre-1970 codes adopted a strength-based concept that did not enforce the ductility measures and energy dissipation capacity of the structure. The lack of appropriate reinforcement details in the frame columns, beams, and joints led to low shear capacity of the beams and columns, and hence, nonductile strength deterioration when shear capacity was reached. Therefore, strengthening such structures is essential and cannot be neglected. FRP composite materials have received increasing attention in the past decades as a potential method for rehabilitation or strengthening existing

structures due to their high strength, light weight, and ease of application (Galal and El-Sokkary 2008).

As a consequence of the recent terrorist attacks, FRPs have been considered as an alternative to reinforce structures to resist blasts. Different from earthquakes, blast loads are high energy impulses that produce high strains in a very short time. Masonry walls subjected to blast loads will exhibit a sudden and severe failure, producing injuries from wall fragments that usually exceed those produced by the blast itself (Urgessa and Maji 2010). This is due to a weak response in the out-of-plane direction of the wall. The use of FRPs can improve the behavior of the wall in this direction, which is an example of how FRPs are a great option to retrofit structures to resist these types of loads.

This report contains a review of several studies conducted in different countries and documents the state-of-practice of the use of FRPs to strengthen structures subjected to dynamic loads. The primary dynamic loads discussed are seismic loads and blast loads. The report is divided into three main sections, reinforced concrete members and structures, masonry walls, and steel members. At the end of the report, the main conclusions are discussed, and recommendations for future work are presented.

2 Reinforced Concrete

Fiber Reinforced Polymers are frequently used to retrofit and repair reinforced concrete structures. Most of the work available in the literature is for reinforced concrete (RC) elements subjected to static or pseudodynamic loads that simulate a seismic event. A typical application for FRPs in reinforced concrete structures is to strengthen buildings designed using preseismic design codes. This is mainly due to the fact that pre-1970 codes adopted a strength-based philosophy that did not consider ductility measures and energy dissipation capacity of the structure. The lack of appropriate reinforcement details in the frame columns, beams, and joints led to low shear capacity of the beams and columns, and hence, nonductile strength deterioration when shear capacity is reached (Galal and El-Sokkary 2008). FRPs also represent a viable option to repair structures that have suffered damage caused by seismic events. Elements repaired with FRPs can regain or exceed their original load capacities when properly repaired.

Recent events have led researchers to investigate how to upgrade important structures to resist blast loads. These types of loads were not incorporated in previous design codes. FRPs are a viable alternative to achieve this goal, but experimentation is not trivial. Malvar et al. (2007) provide a review of the most significant work related to the use of composite materials to resist blast in buildings. This article includes the significant amount of work performed by the authors in the area of concrete columns plus work on beams, walls, slabs and masonry stud walls.

Committee 440 of the American Concrete Institute (ACI) developed guidelines for the design and strengthening of concrete structures with FRP. Document 440.2R-08 (ACI 2008) provides design guidelines for the use of FRPs as external reinforcement for concrete members. The document establishes certain considerations when dealing with seismic design, but they are limited. Section 9.2.3 establishes that most of the available work is related to columns, but limited information is available for frames. It also specifies that for slabs and beams, the location of the plastic hinge formation must be checked to ensure the hinge formation is away from columns and joints. Chapter 10 (Flexural Strengthening) begins by stating that the guides presented do not enhance the flexural strength of members in the expected plastic hinge regions of ductile moment

frames resisting seismic loads. Section 11.1 states that, for elements resisting seismic loads, complete wrapping of the section is needed for shear strengthening. Section 12.3, regarding to the ductility of members subjected to compression or flexural and compression, FRP jackets should be designed to provide a confining stress sufficient to develop concrete compression strains associated with the displacement demands.

This chapter presents work in structural members, i.e., beams, columns, beam-column joints, RC walls, and slabs. Limited work on frames and structures is also presented at the end of the chapter.

2.1 Concrete beams

It is common to strengthen the capacity of flexural members with FRP for both flexure and shear. Extensive work has been developed in this area for static or quasistatic loads (ACI 2007). Work that addresses blast resistance is more limited. This can be explained by the fact that columns and load-bearing walls are more critical when subjected to blast forces, because their failure can produce progressive collapse, rather than beams in which their failure produces only localized failure (Malvar et al. 2007). However, there are a few studies that focused on the use of FRPs to retrofit reinforced concrete beams to resist dynamic or blast loads, and their findings are summarized here.

Jerome (1996) tested 72 laboratory-size beams (3-in. by 3-in. cross-section and 30–in. long) of unreinforced and nylon fiber reinforced light-weight concrete that was externally reinforced with one, two, and three layers of a carbon fiber reinforced polymer (CFRP) preimpregnated sheet (prepreg) applied at the tension side. Another group of specimens had CFRP applied to the tension side and the sides perpendicular to the tension side. The concrete was characterized by compression and indirect tension (splitting tensile test), both quasistatically and dynamically. Sixteen of the beams were subjected to a static three-point bending configuration to establish failure modes and compute failure energies. The rest of the 54 beams were loaded by drop weights in a three-point bending configuration. The hammer weighed 96.14 lb, and the drop heights varied from 2 in. to 24 in. for the beams reinforced on three sides. In the study, the author found that the nylon fibers in the concrete produced little effect on the behavior of the beams compared to beams without the concrete fibers. The beams that performed the best were the ones with the three layers of FRPs. Except for the plain concrete beam, it was found that the fracture energies of the

quasistatic beams were higher than the energies for the dynamically tested specimens. Failure modes from the dynamic and quasistatic loadings were very similar. In addition to the experimental program, this research also included a computational study of the beams by means of section analysis and finite element analysis. The section analysis included three regions, Region 1 – elastic, Region 2 - crack in the tension concrete, and Region 3 - crack in tension, inelastic compression of the concrete, and CFRP elastic. This analysis provided a good prediction of region 1, but overpredicted the responses in Regions 2 and 3. The dynamic analysis, using a half sine wave load, produced higher displacements than the experimental results. A finite element analysis model was performed in ADINA. The analysis was performed on the plain concrete bean and the CFRP reinforced beam with three layers of FRP at the bottom only. The analysis considered the concrete nonlinearity properties using a hypoelastic model. Failure envelopes were included to identify crushing or cracking of the concrete. CFRP was modeled as a linear-elastic material, and the epoxy layer was not modeled. The CFRP layer was modeled as truss members attached to the bottom nodes of the concrete elements. The analysis provided a good prediction on the location and timing of cracks. It also predicted the localized displacement behavior of the impulsive load and the traveling plastic hinge phenomena.

Some of the early work on reinforcement with composite materials for structures subjected to blast loads was conducted by Barbero et al. (1997). The purpose of their investigation was to find new approaches for retrofitting structures against threat environments in order to protect people from direct explosions, fragmentation, and projectile penetration. Specifically, they established that the main effects of blast pressure are the degradation of the integrity of the structure, which may provoke breach or collapse of the structure and spall of interior walls that turn into projectiles.

During the time of the development of the Barbero et al. study, there was little information and available design guidelines for bonded reinforcement to resist blast loads. For this reason, they used data from static tests that were available in the literature to assess the feasibility of reinforcing structures to enhance performance under blast loads. They found that the majority of tests were conducted on reinforced concrete (RC) beams and columns to strengthen them to resist static and earthquake loads.

In their review, they discuss results from work by Faza et al. (1994), Meir and Winistorfer (1995), and Javed (1996). These studies were characterized by the use of two longitudinal plies of carbon fiber on the bottom side of RC beams that resulted in a ductile failure mode and significant gain in bending strength. From these studies, it was observed that different modes of failures occurred for beams under bending loads depending on the amount and type of fiber and the fiber architecture. These failure modes were crushing of the concrete on the compression side, shear failure along the neutral axis of the beam, delamination of the composite reinforcement, and finally, failure of the composite reinforcement. These modes of failure were ordered from the most likely to the least likely to occur.

The composites can enhance the behavior of RC beams by providing higher precracking stiffness and delaying crack initiation on the tension side of the concrete due to the confinement effect of the composite (Howie and Karbahari 1995). The use of composites can also delay the yield of steel rebar due the redistribution of the load between the composite and the rebar. Greater bending stiffness after concrete cracking and steel yielding is also expected when using composites. Finally, the use of composites provides an improvement in ductility, which can be defined as the area under a load-deflection curve up to failure.

Finally, in order to account the effectiveness of composite materials to repair a damaged structure, Howie and Karbahari conducted experiments on preloaded precracked beams. When beams were damaged by preloading and subsequently reinforced with composite materials, they observed that the stiffening effect was less than that observed for undamaged beams because the preloaded beams contained cracks. Beams reinforced with composites exhibited a higher ultimate strength (up to 67 percent in Faza et al. 1994) than the control specimens (no composite reinforcement). The increase in energy absorption was significant even for the preloaded precracked beams reinforced with composite materials. Using small amounts of composite materials can increase the strength and energy absorption significantly under static loads. Based on these results, similar positive effects are anticipated for structures reinforced with composite materials subjected to blast pressure loads.

Ross et al. (1997) studied the application of FRP externally bonded to the tension side of concrete flexural structural members to increase the

stiffness and strength capacity when statically tested. In this study, static tests of reinforced concrete beams externally reinforced with CFRP were conducted. Additionally, similar beams were tested dynamically under blast loads. The explosions were generated using ammonium-nitrate-fuel-oil (ANFO). Concrete slabs with CFRP applied to the tension side were also tested dynamically with ANFO.

Four beams were tested statically and six beams were tested dynamically. Two slabs were tested dynamically under blast loads. In general, beams and slabs were reinforced with CFRP oriented at 0^{o} (i.e., aligned with the long axis of the beam. The tensile modulus of the polymer varied from 18 to 20 × 10^6 psi. The beams and slabs were constructed using lightweight, high-strength (LWHS) concrete containing nylon fibers with an overall density of 122 pcf and compressive strength of 12.22 ksi. Concrete beams with dimensions of 8 by 8 by 108 in. were internally reinforced with two #5 rebars and externally reinforced with a three ply (0/90/0) CFRP laminate of 0.0175-in. thickness. Concrete slabs that were 10 ft by 10 ft by 8-in. thick were reinforced with #5 rebars spaced 12-in. center-to-center in both directions with 2 in. of concrete cover from the bottom surface. The slabs were retrofitted with CFRP laminates (0 /± 45/90) that were 0.08-in. thick. Finally, for the beams and slabs, the CFRP was bonded to the concrete using a two-part epoxy adhesive.

The beams were simply supported horizontally and sandwiched between two square cross-section steel tubes. Six beams were tested, i.e., three control beams without CFRP and three beams with CFRP. Then, ANFO was mixed and suspended between two poles directly overhead of the beam midspan.

The slabs were placed and bolted over a set of I-beams attached to a 3-ft-thick reinforced concrete wall. One control slab was without CFRP, and one slab had externally bonded CFRP. The ANFO explosive was placed on a mound of sand positioned in front of the slab with a given standoff at the geometric center of the slab.

After the experiment was concluded, the beams with CFRP survived the initial blast but were cracked as the blast reflected from the test stand and forced the beams in the opposite direction. The CFRP in some cases appeared to be delaminated. The slabs did not have any appreciable residual displacement measured on scratch gages, 1.69-2.25 mm

retrofitted and control, respectively. The CFRP retrofitted slab had some delamination on one edge.

Scott and Mlakar (1998) conducted impulse tests on a CFRP reinforced concrete beam using a nitrogen-gas, hydraulic loading ram. The tests were conducted at the U.S. Army Waterways Experiment Station, known today as the Vicksburg site of the Engineer Research and Development Center. Four tests were conducted on three beams. A quasistatic test was performed on a control beam (no CFRP), a retrofitted control beam, and a CFRP strengthened beam. The dynamic test was conducted on a CFRP strengthened concrete beam. The beams represented a 1:7 scale typical girder of a section 8-in. tall by 5-in. wide. The quasistatic tests showed that the CFRP reinforced beam supported about 50 percent more load than the control beam. The load in the dynamic test reached almost 35 kips in 5 msec. After the dynamic load, the CFRP strip separated completely from the beam, leaving a separation from the concrete substrate rather than the adhesive layer. It was found that the strengthened beam resisted a load that was 130 percent higher than the quasistatic test on the unstrengthened beam and 60 percent more than the strengthened beam. Even though the use of CFRP increased the resistance of concrete members, there are still questions regarding to the dynamic response. This work used a limited number of samples. The authors concluded that a more in-depth study must be conducted to further understand this behavior.

Analytical models have been developed to serve as predictive tools prior to applying the FRP reinforcement in the field. Models were developed to predict the free vibration of beams. These models were limited to impact loads from falling weights and impulses. However, a more complete model was developed by Hamed and Rabinovitch (2005) that was based on dynamic equilibrium, compatibility of deformations between the various structural components (RC beam, adhesive layer, FRP strip), and concepts of higher order theory. As opposed to previous models, the study investigated a strengthened beam response to three types of dynamic loads, impulse load, harmonic load, and a seismic base excitation. The formulation assumed beam theory for the RC beam and lamination theory for the FRP strip, and the materials followed Bernouli–Euler behavior with small deformations. The adhesive layer was modeled as a two-dimensional (2-D) linear elastic continuum with only shear and vertical normal rigidities while the in-plane longitudinal rigidity was neglected with respect to the rigidities of the RC beam and the FRP strip. It was assumed that the behavior of the

various materials was linear, elastic, and independent of loading and strain rates. These assumptions made the model inappropriate for blast loads, which produce high-rate loadings and deformations.

The model was tested on a simply supported concrete beam that had a 2100-mm span length and was reinforced at the tension side with a 120-mm-wide carbon FRP (CFRP) strip. The beam's section was 150-mm wide and 200-mm tall. The beam was preloaded with a 20 kN/m uniform load prior to applying the CFRP. Concrete cracked properties were used and laminate constants for the CFRP. The most relevant loading condition was the seismic loading using the Imperial Valley earthquake Meloland record. The analysis of the three applied loads revealed behaviors of the system that the dynamic equivalent static load formulation did not reveal. These were (a) the location of the peak shear stresses in the adhesive is variable; (b) the CFRP strip will be subjected to compression due to a dynamic reversing process; and (c) the normal stresses in the adhesive also experience a sign reversing process that can lead to delamination. It was found from the harmonic analysis that the dynamic magnification factor used to determine the equivalent static load differed from the value computed from the dynamic response and the values computed from the internal stresses in the beam and in the adhesive layer, thus underestimating the stresses at the adhesive layer.

Baghiee et al. (2009) reported a study that used the technique of detecting dynamic characteristic changes in FRP retrofitted damaged concrete beams. A testing program composed of nine reinforced concrete beam specimens was conducted. All beams were 150-mm wide and 200-mm tall and were tested in a four-point configuration. The longitudinal reinforcement was four bars of 12-mm diameter for six specimens and 16-mm diameter for the other three. Two concrete strengths were used (20 and 48 MPa) and also two stirrup spacings (100 mm and 200 mm). In the test program, nine RC concrete beams were subjected to an increasing static load to introduce cracks. Following each load step, an experimental modal analysis was performed. Six of the nine beams were strengthened with FRP sheets when the load reached approximately half of the failure load, and tests were continued until failure occurred. To identify the modal characteristics of the beams in each load step, methods based on measured modal parameters were utilized. Three methods were used in this study, two of which are based on mode shapes and modal curvatures, Modal Assurance Criterion (MAC) and Coordinate Modal Assurance Criterion (COMAC). The third

method was based on frequency changes. The comparison of the methods showed that the frequency changes were not fully capable of detecting damage and predicting the potency of strengthening. The MAC is subjected to a minimal change when damage or strengthening is introduced to a structure. This factor cannot determine the stiffness changes in each degree of freedom, but it can present information about the overall stiffness change of the structure due to damage or strengthening. The change of stiffness at each degree of freedom of beams evaluated by COMAC and MAC showed that the damage identification of the beam specimens is best described by MAC.

2.2 Concrete columns

A great deal of research has been dedicated to evaluate rehabilitation of columns because their failure can produce progressive collapse of structures. The ACI 440.2R report (ACI 2008) states that, for seismic applications, the focus has been on strengthening columns using FRP jackets. FRP systems provide additional confinement for columns, increasing the compressive strength, reducing the lap splice length, and increasing the shear strength. The increase in shear strength acquired with FRP jackets is also beneficial for blast loads. A column with an adequate shear capacity can resist column failure and building collapse in the event of a blast load (Malvar et al. 2007).

Breña and Schlick (2007) reported an experimental study that evaluated the performance of retrofitted bridge columns constructed similarly to structures designed during the early 1960s in Massachusetts for low to moderate seismicity. The authors focused on the inadequate reinforcement detailing, specifically the use of short lap splices at the plastic hinge zone combined with inadequate stirrup spacing. Six specimens that were 240 mm in diameter and 950-mm tall were constructed and divided into two groups depending on the applied load. Four of the specimens were rehabilitated using FRP jackets of two different fibers, carbon and aramid. The other two were used as a baseline. The purpose of the FRP jackets was to avoid premature failure of the lapped bars after a limited number of post-yield cycles. The reasoning behind this was that the FRP jackets should provide additional concrete confinement, which plays a major role in the strength of lapped splices under cyclic loads because it controls loss of mechanical interlocking. A single layer of fibers were applied over the entire lap length. The fibers in the laminates were oriented in the hoop direction only in order to minimize any contribution to the flexural strength. The

specimens were subjected to cyclic lateral loads with load, horizontal drift, and strain at the splice bars and in the hoop direction of the FRP jacked recorded for each test. Changes in stiffness and damping from the hysteretic response were examined. Models to estimate the stiffness degradation and the effective damping ratio were also developed, and their results compared with the experimental results.

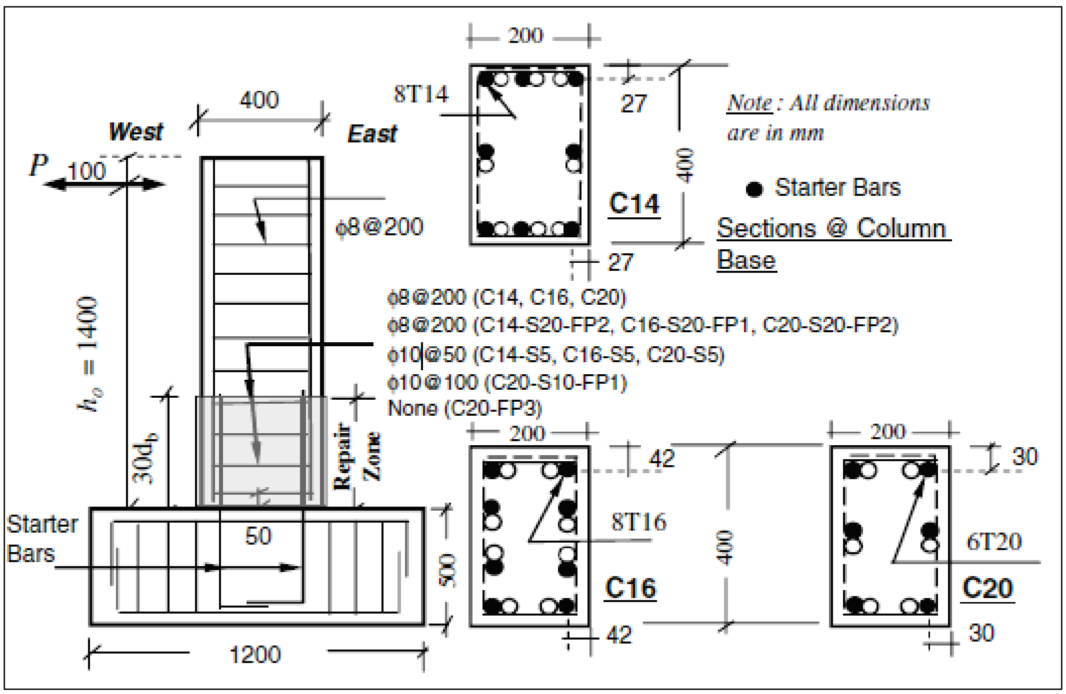
From this study, the authors found that FRP composite jackets are a feasible alternative to rehabilitate bridge columns by maintaining their lateral strength in zones of moderate seismicity. The longitudinal bars were able to yield through the plastic hinge region. Also, their failure mode changed from splitting along the spliced bars to flexural yielding. FRP jackets suffered an increase in hoop deformation as the drift ratio increased. Their integrity was maintained throughout the test, which prevented splitting of the concrete. Damping ratios that are used to represent hysteretic energy dissipation in inelastic reinforced concrete elements and affect the dynamic response of the structural system were determined for the tested specimens. Specimens with FRP jackets obtained higher damping ratios than the control specimens. Finally, stiffness degradation was less in the rehabilitated specimens than in the control specimens. Also, the stiffness degradation was influenced by the axial load ratio, where the specimens with higher axial loads showed an increased amount of stiffness degradation.

Another study that focused on reinforced concrete columns and their critical splice region was conducted by ElSouri and Harajli (2011). These researchers developed a design approach to strengthen the critical splice region of reinforced concrete columns and bridge piers. The approach was based on providing adequate concrete confinement within the splice zone by allowing the spliced bars of the columns to develop enough post-elastic tension strains demanded by seismic events before experiencing splitting bond failure. This approach is consistent with the previously discussed study by Breña and Schlick (2007) in which concrete confinement was used to improve the performance of lap splices. The accuracy of the approach was validated experimentally by evaluating the seismic behavior of full-scale gravity load-designed rectangular columns that were strengthened or repaired in accordance with the approach.

The columns used in the experimental program had a cross section of 200 mm by 400 mm with a height of 1,400 mm. The columns were

reinforced with steel bars varying from 14 mm to 20 mm. In each column, the longitudinal reinforcement was lap spliced at the base with starter (dowel) bars projecting above the footing. The dimensions and reinforcing details are shown in Figure 2.1. The splice length for all the columns was 30 times the diameter of the reinforcing bar (30 d_b). This length of the splice is consistent with the ACI building code (2008) requirements for gravity load design. The strengthening system consisted of providing confinement within the splice zone. Three types of confinement were used and compared, internal steel ties, external fiber polymer reinforced jackets, and a combination of both. The steel ties were grade 60 with a 10-mm diameter. The FRP reinforcement consisted of unidirectional CFRP sheets.

Figure 2.1. Dimensions and reinforcement details of column specimens (from ElSouri and Harajli 2011).



The results showed that all of the as-built columns experienced splitting bond failure of the starter bars before the bars were able to develop their yield strength. This was caused by the inferior bond capacity of the spliced bars associated with small splice length, small concrete covers, and/or inadequate area of transverse steel confinement. The splitting bond failure of the starter bars led to quick stiffness and strength degradations under cyclic loading and considerable concrete damage within the splice region of the columns. When compared with the as-built columns, the results demonstrated that all repaired or strengthened columns developed

yielding of the starter bars at peak lateral load and had significantly less concrete damage within the splice region. It was observed that all repaired or strengthened columns developed full flexural capacity and, hence, failed in a predominantly flexural mode, combined with cyclic bond degradation of the starter bars at large drift reversals.

Morrill et al. (2004) also conducted an investigation focusing on bridge columns. In their study, the focus was on the specific case of hollow rectangular bridge columns retrofitted with CFRP. The scope of the investigation was the development of an analytical model incorporating the effect of CFRP composite straps to predict the lateral loading characteristic for the columns. The results of the analytical model were then compared with experimental results from a series of tests performed on eight column specimens. In order to develop the analytical model, the moment-curvature relationships for rectangular hollow sections of columns wrapped by CFRP were first determined and then the nonlinear lateral load-displacement relationships were obtained accordingly.

The analytical model employed nine different constitutive laws of confined concrete. The predicted results using the different constitutive laws were compared with the experimental results in order to determine which law could better represent the behavior of confined concrete in a hollow section. The nine constitutive laws that were used were (1) unconfined Kent and Park model (1971), (2) confined Kent and Park model (1971), (3) modified Kent and Park model (1971), (4) Mugurama et al. model (1980), (5) Sheikh and Uzumeri model (1982), (6) Mander et al. model (1988), (7) Fujii et al. model (1988), (8) Saatcioglu and Razvi model (1992), and (9) Hoshikuma et al. model (1997). The shear capacity of the columns was also estimated using different approaches in order to determine which one was the more appropriate for a hollow column. For the columns without FRP retrofit, four approaches were used (1) ACI 1995 code provisions, (2) the UCB (University of California, Berkeley) model (Qi and Moehle 1991), (3) the UCSD (University of California, San Diego) model (Kowalsky and Priestly, 2000), and (4) the Caltrans (California Department of Transportation) model (Lehman and Moehle 2000). The shear capacity was calculated point by point along the load-displacement curve since shear capacity varies with the displacement ductility factor. For the columns with CFRP, the UCSD model and the ITRI (Industrial Technology and Research Institute) model (Chu et. al 1998) were used to predict the shear capacity. These two models consider that the total shear

capacity is a function of the shear capacity due to concrete, steel, axial force, and the contribution of the CFRP sheets. The variation of these two models is basically in the contribution provided by the CFRP sheets.

The experimental program for this study consisted of testing eight reinforced concrete hollow columns under a constant axial force and a cyclically reversed horizontal load. The purpose was to investigate their seismic behavior, including flexural ductility, dissipated energy, and shear capacity. The horizontal load was applied at a quasi-static rate in displacement-controlled cycles. Of the eight columns tested, two were not retrofitted with FRP. The shear reinforcement in one of these two columns satisfied the requirements of the ACI 318 1995 code. The other one had only 35 percent of the shear reinforcement required by ACI. The other six specimens were retrofitted with FRP. In two of these retrofitted specimens, the concrete was not confined, while the concrete was confined in the other four retrofitted specimens. The number of layers of FRP sheets used for the retrofitting varied from none to four.

The experimental results showed that the ductility factor increased with the increase of the number of FRP sheets. However, when the number of FRP sheets was large, the increase of the ductility factor was limited. This can be seen in the results for the confined retrofitted columns. The specimen with shear reinforcement and without FRP layers showed a ductility of 5.3. When two sheets of FRP were used, the ductility factor increased to 5.7. When four sheets were used, the ductility factor increased to 6. In the columns without confinement, the FRP has a major effect in ductility. For the unconfined retrofitted columns, the ductility increased from 3.4 for no layers of FRP to 4.9 for two layers of FRP. This means that ductility had an approximate increase of 44 percent because of the FRP layers. It was also observed that the FRP sheets could eliminate the shear cracks and change the failure mode of specimens from shear to flexure.

The experimental results were used to compare the predicted results using the nine constitutive models of confined concrete. It was found that the modified Kent and Park model was in better agreement with the experimental results. Similarly, the experimental results were compared with the shear capacity estimates. For the shear capacity prediction in columns without FRP, it was found that the ACI 318-95 approach was too conservative, while the Caltrans model overestimated the shear capacities. The UCSD approach gave predictions very close to the experimental

results. For the retrofitted columns, it was observed that the experimental shear force was less than those predicted by both the UCSD and ITRI models. The FRP retrofit increased the shear capacity of specimens and changed the failure mode from shear to flexure.

Perrone et al. (2007) studied a strengthening technique for square concrete columns combining CFRP laminates and strips of wet layup CFRP sheets. Their research program also had the purpose of evaluating the influence of concrete compressive strength on load carrying and energy dissipation capacities of strengthened RC columns. The strengthening technique was developed with the purpose of increasing both the flexural and energy dissipation capacities of the columns. To study the effectiveness of this strengthening technique, test specimens were subjected to constant axial compressive loads and increasing lateral cyclic loading.

As part of the experimental program, eight columns with a concrete compressive strength of 8 MPa (1.2 ksi) and another with a compressive strength of 29 MPa (4.2 ksi) were designed and fabricated. Three of the 1.2-ksi columns had different longitudinal steel reinforcement ratios and were not retrofitted. These were used as the reference columns. After performing the testing of the reference columns, they were strengthened with the proposed technique. Therefore, they were strengthened after they had suffered damage. The remaining five 1.2-ksi columns were strengthened before they were tested, hence, being retrofitted when they were undamaged. The strengthening technique used CFRP laminates of 9.37×1.4 mm^2 (0.37×0.05 in.^{2}) cross-section area and strips of CFRP wet layup sheet. The first strip was applied at the bottom of the column with a width of 430 mm (16.9 in.) that is similar to the pre-evaluated length of the plastic hinge, and the remaining strips with a width of 150 mm (6 in.) were applied between the existing steel hoops. The number of CFRP laminates was evaluated to provide an increase of 50 percent of the load carrying capacity of its corresponding reference column when strengthened in the undamaged state.

For the columns with low concrete strength (1.2 ksi) and strengthened after they were tested and damaged, the results showed that the technique provided an average increment of 46 percent in terms of load carrying capacity. The number of CFRP laminates that were applied was intended to provide an increment of 50 percent for the load carrying capacity of

columns retrofitted when they were undamaged. Since the increment of 46 percent is relatively close to 50 percent, it can be said that this technique is also effective for increasing the load carrying capacity of columns with significant damage. In contrast, the authors concluded that due to the low concrete strength, the benefits of the strengthening technique for the energy dissipation capacity of columns retrofitted after suffering damage were marginal. They based this conclusion on the fact that, in general, the longitudinal bars did not yield. However, in the case of the columns retrofitted when they were undamaged, both the column load carrying capacity and energy dissipation capacity saw considerable increases. On average, the load carrying capacity increased 67 percent, and the energy dissipation increased 56 percent. For the column of moderate concrete compressive strength (4.2 ksi), the performance of the technique was even better, since an increase of 39 percent and 109 percent was obtained for the load carrying capacity and energy dissipation capacity, respectively, when compared to the corresponding column of 1.2 ksi.

Realfonzo and Napoli (2009) evaluated the seismic performance of 24 samples of RC columns that were externally confined with fiber-reinforced polymers (FRPs). The objective of their study was to evaluate the effectiveness of confining systems made of FRP in enhancing ductility and energy dissipation capacity of RC columns belonging to Gravity Load Designed (GLD) existing buildings. Specimens consisted of full-scale square columns of 300 by 300 mm (12 by 12 in.) cross section with a length of 2,200 mm (86 in.). The columns were designed to be representative of Gravity Load Designed RC Frame in existing buildings in Italy. For this reason, they are reinforced by using both smooth and deformed longitudinal steel rebars. Such columns were subjected to a constant axial load and a monotonic or cyclic flexure. Tests were conducted in displacement control on both FRP confined and unconfined RC columns. Two confinement systems were used in the study. The first used two or four wrapped unidirectional carbon (CFRP) or glass (GFRP) layers around the member. In the other confinement system, four longitudinal steel angles were placed in the column corners before applying CFRP layers around the column. The steel angles were glued to the concrete substrate using an epoxy adhesive. In some cases, these steel angles were anchored to the foundation by means of steel connectors.

The authors drew several conclusions based on the results of their experimental investigation. Regardless of the axial load value, the

confinement through FRP resulted in a significant increase in ductility. When the axial load was high, the FRP confining action also led to a non-negligible improvement in strength. The specimens with longitudinal steel angles anchored to the foundations showed an increase in flexural strength when compared to members strengthened with FRP only. Columns with steel angles that were not anchored to the foundation provided a higher strength and ductility when compared to results of columns confined with only FRP. It was observed that the stiffness degradation is independent of the presence of the FRP confinement system. Also, columns reinforced with smooth rebars showed higher stiffness degradation than the corresponding ones with deformed rebars. In terms of the total energy dissipated, it was observed that the columns strengthened with FRP systems dissipated much more energy that the non-strengthened columns.

Saiidi et al. (2003) conducted a study of reinforced concrete bridge columns with structural flares. The objective of the study was to determine an appropriate retrofit method for columns with inadequate shear capacity under earthquake loads. Four 1/3rd scale models were tested on a shake table to failure, one representing as-built columns and the other three representing columns retrofitted with jackets. The prototype represented the critical columns of a 16-span viaduct in Reno, NV. The bridge was designed in 1979 according to the 1977 AASHTO Standard Specifications (AASHTO 1977) and the 1978 Interim Specifications and was constructed in 1981. One model represented the as-built column and the other three models represented the retrofitted columns. One model represented a column retrofitted with a steel jacket, a second column with a glass FRP jackets, and the third a column with a carbon FRP jacket. All the retrofitting jackets were designed to maintain the plastic hinge location away from the connections. In the model with steel jackets, this was accomplished by leaving a gap in the jacket where the plastic hinge was expected. The way of controlling the plastic hinge in the models with FRP was by reducing the number of fabric layers at the plastic hinge location.

When using FRP to improve shear strength and confinement in columns, the fabrics should be placed so that the fibers are horizontal. Vertical fibers should be avoided because they would increase the flexural strength and attract larger shear forces. In flared columns, the fibers in a continuous wrap will no longer be horizontal as they bend around the corners. The slope of the fibers will increase with every turn, providing a vertical force component and, hence, increasing the flexural capacity. In order to

prevent this, a new method of fabric installation was developed as part of the study. The column was covered with a series of butted U-shaped straps with overlapping ends.

The results proved that leaving a gap in the jacket was successful in controlling the plastic hinge location and avoiding damage to the element end. It was confirmed that the method of installing FRP composite fabrics in the form of U-shaped straps developed by the authors was an effective means of constructing composite jackets for flared (or other nonprismatic) members. All the jackets that were tested accomplished their primary goal of enhancing the shear and displacement ductility capacity of the columns with structural flares. The jackets changed the mode of failure from shear/flexure to flexure. The seismic performance of the steel-jacketed column was very similar to that of the columns with FRP jackets. There was no clear advantage of any of the jacket types over others with respect to structural performance. The advantage of the FRP jackets over steel jackets is that they are easier to construct in relatively tight areas, which are common in bridges.

Shan et al. (2006) analyzed the residual performance of FRP-retrofitted columns damaged by simulated seismic loading. Eight model columns with a shear aspect ratio of 5.0 were tested first under cyclic lateral force and a constant axial load equal to 20 percent of the column gross axial load capacity. After being damaged by lateral cyclic loading, five of the model columns were subjected to long-term axial loading to study the residual performance of damaged columns. For the long-term axial load test, the columns were initially loaded at an axial load equal to 0.2 A_g fć and observed for 30 days. The axial load was then removed and a higher load of 0.4 A_g fć was reapplied for a period of 60 days.

The specimens were designed as vertical cantilever columns with a strong footing. They were circular columns with a diameter of 375 mm (15 in.) and height of 1,500 mm (60 in.). Twelve longitudinal reinforcing bars with a diameter of 12 mm (0.5 in.) were evenly arranged in a circle with a clear cover of 20 mm (0.8 in.). For all specimens, the transverse reinforcements were 6-mm (0.23-in.) diameter spirals with a spacing of 60 mm (2.4 in.). This corresponds to 44 percent of the required transverse reinforcement of the seismic design standards of the ACI 2002. Glass fiber-reinforced plastic (GFRP) and carbon fiber-reinforced plastic (CFRP) sheets were both used for retrofitting. Four layers of CFRP or five layers of GFRP were

wrapped for a 400-mm- (15.7-in.-) long segment near the column bottom end. The 400-mm segment length was approximately the same length as was the diameter (375 mm) of the column. In this length is where the transverse reinforcement needs to be provided with seismic design provisions. The next upper 400-mm portion was also wrapped with two layers of FRP jacket to prevent possible shear failure.

Similar to other studies of columns wrapped with FRP, this study showed that the capacity and ductility of the RC columns can be effectively increased with the wrapping of an FRP jacket when transverse reinforcement is not sufficient. A different finding from this study is that during long-term axial loading test after being subjected to some degree of lateral loading damage, the deformation of the FRP retrofitted columns was lower when compared with an unretrofitted column subjected to a similar loading condition. The development of long-term loading deformation of the retrofitted column was found to be related to the previous damage level and the modulus of elasticity of the FRP. The post-damage long-term axial strain corresponding to a time increment can be estimated with reasonable accuracy using the simple creep model recommended by ACI Committee 209, so long as the initial strain is calculated using the elastic strain amplified with a suggested correction factor. The study essentially confirmed that within the testing range of the axial load ratios between 0.2 and 0.4 A_g fć, the long-term axial loading applied after the column suffered damage would not cause any significant creep effects to the FRP retrofitted columns.

Wu et al. (2008) conducted an experiment providing reinforcement in the middle of the straight sides of rectangular columns. The reasoning behind this is that jacketing is less effective in large square/rectangular RC columns due to the inability of the jackets to confine the concrete in the middle of the column sides. The authors propose a retrofit method in which small FRP bars are inserted into the concrete in the plastic hinge zone. The inserted FRP bars act as horizontal reinforcement to increase the ductility of the concrete. This reinforcing technique was combined with the conventional jacketing to effectively confine the concrete in all parts of the cross section.

The specimens that were tested represented part of a bridge column or a building column from the section of the maximum moment to the point of contra-flexure. Six half-scaled columns were tested, and the test results

demonstrated the effectiveness of the method. One of the specimens was a non-retrofitted column used for monotonic load test as a control model. The second purpose of this monotonic test was to find the yield displacement of the non-retrofitted column that was required to determine the cyclic loading history. Another non-retrofitted specimen was used for cyclic test as the benchmark for comparison with other retrofitted and cyclically loaded columns. Three columns were retrofitted by different schemes. One of them was retrofitted by glass FRP (GFRP) bar embedment. The other two columns were retrofitted with carbon FRP jacketing plus the identical glass FRP bar embedment as that of the other retrofitted column. The last column was retrofitted by jacketing only without bar embedment.

The test results demonstrated that the horizontal bar embedment increased the ductility of the columns, proving that the proposed retrofitting method was effective. When the stiffness degradation was observed, it was noticed that all the retrofitted columns had similar stiffness degradation. This indicates that the bar embedment was as effective as jacketing in confining the concrete and maintaining the integrity of columns. It can be concluded that this method was effective in delaying the concrete deterioration, preventing buckling of longitudinal reinforcement, and increasing the ductility and energy dissipation of the retrofitted columns.

Instead of focusing on seismic loads, Morril et al. (2004) studied retrofitting concrete columns and walls to protect buildings due to blast loads. The idea is to avoid a possible progressive building collapse induced by explosive loads. To accomplish this, they presented a design procedure developed by Karagozian and Case (K & C) in 1973, using steel jackets and FRP wraps to successfully prevent column damage and building collapse. This design procedure is applicable for circular, square, and rectangular columns. The Column Blast Analysis and Retrofit Design (CBARD) procedure was based on results of first-principle calculations that were verified and validated against many full-scale blast tests as well as quasi-static laboratory tests. Similarly, a wall procedure was also developed and presented. Both procedures were coded into a MS Windows program to complete designs of columns and walls. The paper details the design (CBARD) procedure on which the program is based and then describes a series of tests that were used to verify and validate it.

The retrofit design procedure was based on the guidance contained in three design manuals, i.e., DAHS (DTRA, 1998), P-397 (NAVFAC, 1991), and ACI 318 (ACI 2002) Morril et al. (2004). Then, a simplified engineering model (a single degree of freedom or SDOF model) was employed to develop the responses needed to select the design parameters. The first step is determining the response of non-retrofitted structural components to the design threat. If some element fails due to the loads, the procedure's program has the option to determine the required thickness of the steel jacket or composites wraps needed to resist the thread loads.

In the program model, the user first enters the information about the element to be retrofitted, such as geometry of the component, concrete properties, reinforcement properties, retrofit type (steel jackets or FRP) and its thickness, and design threat. For a given load, the SDOF model will calculate a displacement-time history and corresponding strain rates in the materials.

It is important to input the concrete shear capacity without a factor of safety, because if the shear capacity of the section is not enough to induce a flexural failure, there will be a brittle failure. To obtain the shear capacity, the authors used ACI 318. For this reason, CBARD computes the flexural resistance function. The flexural resistance function is a plot of Flexural Resistance vs. Midspan Deflection (analog to a Moment vs. Curvature plot).

Therefore, the process of design is to compute the resistance function of the element and its shear capacity. If the shear capacity is insufficient, the flexural mode will not occur, and the energy dissipation will be reduced. For columns, this deficiency can be easily eliminated by wrapping the column with a steel jacket or composite wraps. To consider this, CBARD has in its input menu the options to enter the steel jacket thickness or number and type of composite wrap to be used. The resistance function is recalculated by the program. Now, the user needs to increase the steel jacket thickness, or number of FRP wraps, either necessary, until sufficient diagonal shear capacity is provided.

The CBARD code was validated for RC columns against results of several full-scale blast tests using full-scale buildings or full-scale components in a testing frame. Studies showed that steel jackets and composites wraps can provide sufficient diagonal shear enhancement to make the column

survivable. Retrofitted and non-retrofitted column predicted deflections using CBARD compared favorably with blast test experimental results. In conclusion, this simple Windows-based code can predict the resistance of an existing RC columns and walls to blast and design the retrofit necessary to survive the blast load.

2.3 Beam-columns joints

The research on beam-columns joints strengthened with FRP focuses on the effects of seismic loads. Beam-column joints are considerably affected by seismic events; therefore, their design is a key detail for structures prone to seismic loads. The main concerns with beam-column joints under seismic loads are poor shear strength and nonductile detailing of the joint.

Al-Salloum and Almusallam (2007) and Almusallam and Al-Salloum (2007) discuss the seismic response of interior reinforced concrete beam-columns joints upgraded with FRP sheets. In the first paper, the authors present the results of an experimental study on the efficiency and effectiveness of carbon reinforced polymers in upgrading the shear strength and ductility of seismically deficient beam-column joints. In the second paper, a procedure for the analytical prediction of shear strength of interior beam-column joints strengthened with externally bonded FRP sheets is presented.

For the experimental study (Al-Shalloum and Almusallam 2007), four reinforced concrete interior beam-column sub-assemblages were constructed with nonoptimal design parameters, inadequate joint shear strength with no transverse reinforcement, representing the preseismic code design construction practice of joints. Six specimens were tested: two control or baseline, two strengthened, and two repaired. The two baseline and two strengthened specimens were constructed using two different schemes. Figure 2.2 shows Scheme 1 that consisted of CFRP sheets epoxy bonded to the joint, the beam, and part of the column regions. Figure 2.3 shows Scheme 2 in which sheets were epoxy bonded to the joint region only and mechanical anchorages were employed to effectively prevent against any possible debonding. To apply the mechanical anchorage system, three through holes were drilled on either side of the epoxy-bonded CFRP upgraded joint, and bolts were passed through each hole. All four subassemblages were subjected to cyclic lateral load histories to provide the equivalent effect of severe earthquake damage. Further, the two control specimens that were damaged during testing were repaired by

filling the cracks with epoxy and wrapping the joints with CFRP sheets following the two schemes. These repaired specimens were then subjected to the cyclic lateral load history.

The experimental study confirmed that externally bonded CFRP sheets can effectively improve both the shear strength and ductility of beam-column joints. The degree of improvement was affected by how the CFRP sheets were attached to the joint and whether or not the mechanical anchorage was used. Scheme 1 is an efficient scheme, because it upgrades both the joint and the beam. Due to the absence of any mechanical anchorages in Scheme 1, debonding (bulging) of externally bonded CFRP sheets occurred at higher stages of loading, which allowed cracks to form and widen under the fiber sheets. Scheme 2 is an economical and effective scheme for joint strengthening, and the CFRP sheets were applied in such a way that the possibility of debonding is eliminated. When Scheme 2 is used, the increase in capacity of the joint causes the failure to be redirected to the beams. The CFRP sheets also made the joint stiffer against distortion.

Figure 2.2. Schematic representation of Scheme 1 (Al-Salloum and Almusallam 2007).

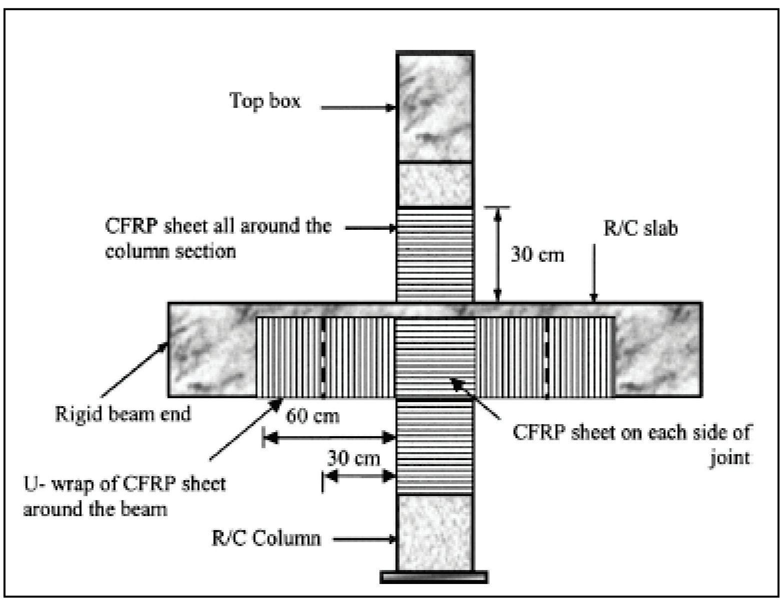
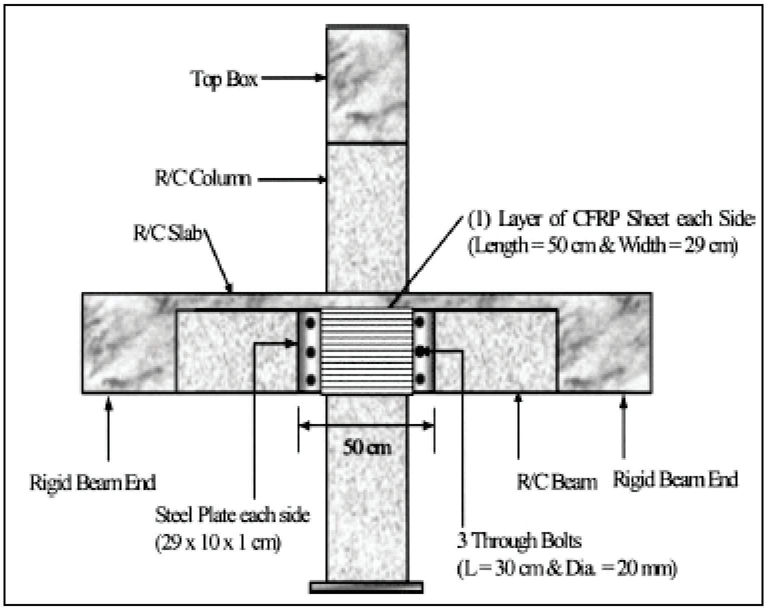


Figure 2.3. Schematic representation of Scheme 2 (Al-Salloum and Almusallam 2007).



The second study (Almusallam and Al-Salloum 2007) developed a comprehensive computer program that was developed to compute the shear capacity of a repaired joint. It also traces the state of stress and strain in the joint until failure. Analytical predictions were made for shear capacity, diagonal tensile stress, and other governing parameters for FRP-strengthened RC-beam column joints. To make these analytical predictions, the joint was idealized as a three-dimensional (3-D) element. Consistent with the experimental specimens, two types of specimens were modeled, one without mechanical anchorage (Scheme 1) and one with mechanical anchorage (Scheme 2). The analytically predicted shear capacities and joint shear stress variations for the control and FRP strengthened beam column joints were compared with the experimental observations. The following assumptions were made in the predictions: shear stress distribution was uniform over the boundaries of the joint; the joint was already loaded at the time of strengthening, and therefore, a set of initial normal and shear strains existed; and strengthening of the joint occurred through the use of unidirectional FRP sheets placed vertically and horizontally.

The predicted shear capacities and joint shear stress variations for the control and the FRP strengthened beam-column joints were in good agreement with the experimental observations, which provided confidence in the validity of the analytical model. The mathematical formulation in the model was extended further to compute diagonal tensile stresses in the

control and FRP strengthened joints. The predicted values of diagonal tension were also in good agreement with experimental. The results showed that the use of externally bonded composite sheets substantially increased the diagonal tension capability of the joint. The magnitude of this increase for Schemes 1 and 2 for the control specimens were 23 percent and 66 percent, respectively. It was observed that as the quantity of FRP increased, confinement increased, which in turn increased the shear strength of the joint. As expected, it was also observed that shear capacity of the joint increased with increases in axial load.

The experimental research was extended in 2010 by two separate studies of exterior and corner RC beam-column joints (Alsayed et al. 2010). In each study, four joint specimens (two control and two retrofitted) were constructed based on preseismic code design construction practice. Similar to the previous studies, two schemes for reinforcing the joints were investigated. In Scheme 1, CFRP sheets were epoxy bonded to the joint, the beams, and part of the column regions. In Scheme 2, CFRP sheets were bonded to the joint region only and were provided with mechanical anchorages. All subassemblages were subjected to cyclic lateral, quasistatic load histories. The damaged control specimens were then repaired by filling their cracks with epoxy and externally bonding them with CFRP sheets using the two mentioned schemes. These repaired specimens were subjected to similar cyclic lateral loads.

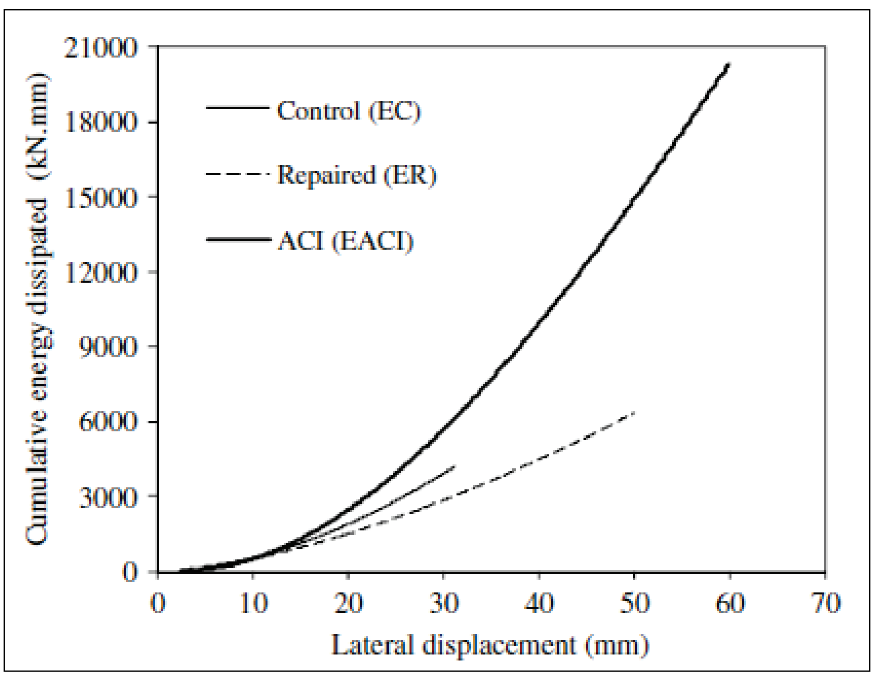
The results with exterior and corner beam-column joints were consistent with the previously discussed results for interior beam-column joints (Al-Salloum and Almusallam 2007). The test results demonstrated that externally bonded CFRP sheets improved the shear strength and deformation capacity of the joints. The magnitude of effectiveness is dependent on how CFRP sheets were attached to the joint and whether or not mechanical anchorages were used. Due to the absence of any mechanical anchorages in Scheme 1 and at higher stages of loading, debonding of externally bonded CFRP sheets occurred, which allowed cracks to form and widen under the fiber sheets. In Scheme 2, debonding wasn't an issue, and it was observed that failure was directed to the beams.

In 2011, this same group of researchers compared the efficiency and effectiveness of carbon-fiber-reinforced polymer (CFRP) sheets for upgrading the shear strength and ductility of a seismically deficient exterior beam-column joint with an American Concrete Institute (ACI)-based design

joint specimen (Al-Salloum et al. 2011). One as-built joint specimen, representing the preseismic code design and construction practice for joints, and one ACI-based design joint specimen, satisfying the seismic design requirements of the current code of practice, were cast. The as-built joint specimen was used as a control specimen. These two specimens were subjected to cyclic lateral load histories to induce damage equivalent to damage expected from a severe earthquake. The damaged control specimen was then repaired by filling its cracks with epoxy and externally bonding CFRP sheets to the joint, the beam, and part of the column regions, similar to Scheme 1 in their previous studies. This specimen was identified as the repaired specimen. The repaired specimen was subjected to a similar cyclic lateral load history, and its response was recorded.

Figure 2.4 shows a comparison of the cumulative energy dissipation versus the lateral displacement of all specimen types. The energy dissipation ability of an ACI-compliant beam-column joint is substantially higher than that of an as-built beam-column joint. The use of CFRP sheets can increase the energy dissipation ability of an as-built exterior joint by as much as 40 percent.

Figure 2.4. Cumulative energy dissipation by the as-built control (EC), repaired (ER) and ACI-based (EACI) specimens (Al-Salloum et al. 2011).



The authors concluded from the results that externally bonded CFRP sheets can effectively improve both the shear strength and deformation capacity of seismically deficient beam-column joints to an extent comparable to ACI-based design joints. The increase in the peak load and deformation capacity by CFRP upgrading is dependent on the number of CFRP layers used in the repair. It is possible for CFRP upgraded specimens to exceed the ACI-based beam-column joint ultimate load value with the use of more layers. Note that increasing the number of layers changes the stiffness of the system. It should be verified that this change in the stiffness of the system does not adversely affect the load sharing among the members resulting in an additional load that causes early debonding without the development of CFRP strength.

Joints without adequate reinforcement were also investigated by Ilki et al. (2011). In this case, they studied the behavior of eight full-scale beam-column joints built with plain bars and low-strength concrete. No transverse reinforcement was present in the joint cores. A first series of tests was conducted on three control specimens to investigate the behavior of joints before FRP retrofitting. A second series of tests was conducted on five specimens to investigate the behavior of FRP retrofitted joints. As seen in Figure 2.5, the hooks of top longitudinal bars of the beam were welded to the hooks of the bottom bars in five of the specimens to investigate different anchorages of the beam bars and whether this technique could be utilized to retrofit joints with insufficiently anchored beam bars in the joint.

Figure 2.5. Welding of hooks of top and bottom longitudinal beam bars and replacing the low-strength concrete with high strength cement-based repair mortar (Ilki et al. 2011).

The specimens were tested under the combined action of constant column axial load and static lateral displacement reversals acting on the tip of the beam. It was observed that the FRP-retrofitted specimens with welded hooks at the beam longitudinal bars achieved the flexural capacity of the

framing beam, but the control specimens and FRP-retrofitted specimen without the welding of hooks of beam longitudinal bars could not reach the flexural capacities of the framing beam and columns. The use of FRP only was not sufficient to prevent slippage of the beam longitudinal bars. In a specimen without FRP but with the welding procedure, the strength was governed by flexural capacity of the beam, and the specimen could keep its strength until 4 percent drift ratio. After the 4 percent drift, the shear damage in the joint caused a sharp decrease in the strength of the specimen. When the joints were retrofitted with FRP sheets, the strength decay was significantly retarded. The specimens kept their strength until the drift ratios reached approximately 9 to 10 percent.

A rehabilitation scheme for repairing moderately damaged reinforced concrete beam-wide column joints was proposed by Li and Kao (2011). In order to prove the rehabilitation scheme, four nonseismically detailed interior beam-wide column joints were constructed and tested. Two of these specimens had a column-to-beam width ratio of approximately 3.56. The other two specimens had a column-to-beam width ratio of approximately 7. All four subassemblages were subjected to similar cyclic lateral displacements to provide the equivalent of severe earthquake damage. The damaged control specimens were then repaired by following two rehabilitation schemes. In both schemes, a layer of FRP L-wrap was applied at each of the four corners of the joint. Scheme 1 consited of a layer of continuous GFRP sheet applied in the direction parallel to the beam axis. Scheme 2 consisted of a layer of continuous CFRP in the direction parallel to the beam axis. These repaired specimens were then retested and their performance compared with that of the control specimens.

The test results showed that for the repaired specimens with a column-to-beam width ratio of approximately 3.56, both schemes were able to recover the seismic performance of the original specimens. However, none of the schemes were able to recover the seismic performance of specimens that had a column-to-beam width ratio of approximately 7. Also, for the specimens of width ratio of 7, Scheme 1 was not as effective as Scheme 2. The authors suggested that to further improve the effectiveness of the rehabilitation of these specimens, additional CFRP layers should be provided in the repair scheme.

2.4 Walls

The amount of research focusing on the use of FRPs to strengthen reinforced concrete walls is limited. RC structural walls are often used in buildings to resist lateral loads, such as earthquake loads. Buildings containing shear walls exhibit fair earthquake performance (Li and Lim 2010). This may be a cause for the limited amount of research focusing on the use of FRP to retrofit RC walls for seismic loads. Similarly, there is limited amount of research for the use of FRP to retrofit walls under blast loads.

Li and Lim (2010) conducted an investigation focusing on the fact that older shear walls that were designed with earlier seismic design provisions have suffered extensive damage from major earthquakes, leading to the use of FRPs as a repair and strengthening method for these walls. The authors conducted an experimental study on the seismic performance of axially loaded reinforced concrete walls with boundary elements confined by limited transverse reinforcement. Four specimens were tested, two with a height of 7.4 ft representing low-rise buildings and two with a height of 11.5 ft representing medium-rise buildings. These wall specimens were cantilever walls, with a steel beam at the top for applying the vertical and the in-plane horizontal loading. The walls were subjected to a constant axial load. Cycles of quasistatic cyclical loading were applied for the in-plane horizontal loading. For all specimens, the web reinforcement composed of a double orthogonal grid of 10-mm- (0.393-in.) diameter bars spaced at 250 mm (9.8 in.). The boundary elements were rectangular and had a cross section of 300 mm (11.8 in.) by 150 mm (5.9 in.). These boundary elements were reinforced with eight bars of 10-mm (0.393-in.) diameter. The walls were subjected to axial compression loading and cyclic lateral loading until the wall specimens failed. The failure mode was similar for all four specimens. Hairline shear and flexural cracks were formed along the height of the specimens, while heave damage was concentrated at the base. The wall specimens failed in a predominantly flexural mode characterized by concrete crushing and reinforcement buckling at the confined edges.

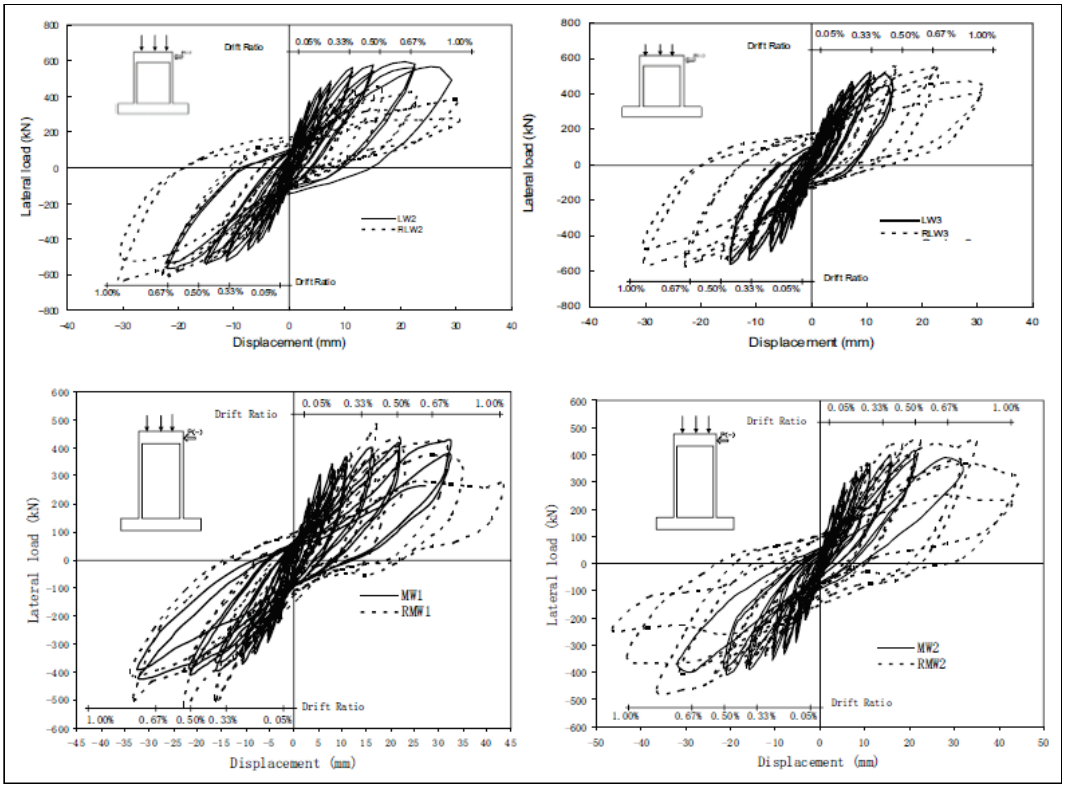
Following the loading and failure of the original wall specimens, the walls were repaired by a combination of high-strength mortar, epoxy injection, and a FRP wrapping scheme. The high-strength mortar was used for replacing heavily cracked and spalled concrete that was removed from the wall base. Cracks at the base of the specimens were repaired by injecting

epoxy. Subsequent to repair, specimens were strengthened with FRP. One low-rise building specimen and one medium-rise building specimen were wrapped with CFRP. The remaining two specimens were wrapped with GFRP. GFRP anchors were placed at various locations along the development length of the sheets. The anchor consisted of a bundle of fibers impregnated with epoxy in drilled holes in the wall. The embedded length of the anchor from the surface of the wall had a minimum length of 50 mm.

The test results showed that the shear strength of the repaired walls increased and shear cracking was effectively controlled by much reduced shear displacement. As may have been expected, the use of CFRP is more effective than GFRP in recovering the strength of repaired components. In general, it was observed that the dissipated energy of the repaired specimens were significantly larger than those of the original counterparts for both the low-rise and medium-rise walls, as observed in the hysteresis loops in Figure 2.6. The stiffness of the repaired specimens was on average higher than that of the original specimens at higher drift ratios. It was also observed that the L-shaped anchor strips were effective as there was no debonding observed on the main body of the wall in all repaired specimens. However, debonding of the L-shaped strips occurred at the base of the wall.

Mutalib and Hao (2011) also made a study of reinforced concrete walls strengthened with FRP. In this work, the authors wanted to analyze the structural response under blast loads of FRP strengthened walls with or without additional anchors. The authors used the commercial software LS-DYNA to calculate responses of nonretrofitted and FRP strengthened RC walls under blast loadings. In the model, the FRP was attached to the rear face of the wall for the FRP to act in tension. Since the boundary conditions are important to the analysis, the concrete slab, side walls, middle column, and roof were also included in the model to represent the actual structure. Three different configurations of FRP-strengthened RC walls were investigated, FRP-strengthened RC wall without anchors, FRP-strengthened RC wall with anchors at the boundary, and FRP-strengthened RC wall with distributed anchors. The concrete-anchor connection was modeled using a tied node. The contact between the FRP nodes and concrete surface at bolt locations was modeled using the contact “tiebreak nodes to surface” in LS-DYNA. Eight-node solid elements were used to model concrete walls. Each node had six degrees of freedom.

Figure 2.6. Hysteresis loops of original (solid lines) and repaired (dotted lines) wall specimens (Li and Lim 2010).



The authors found that FRP strengthening effectively increased RC wall blast loading resistance capacity. It was noted that bond strength played a significant role in maintaining the composite action between the FRP and concrete. The average maximum displacement for FRP retrofitted walls with bond strengths of 0.4 ksi, 0.7 ksi, and 1.45 ksi decreased by 17.2 percent, 20.1 percent, and 24.9 percent, respectively, as compared to the nonretrofitted wall. For the FRP retrofitted wall without anchors, the one with 0.4-ksi bond strength exhibited a higher degree of delamination, while the one with 1.45-ksi bond strength had the least delamination. The authors concluded that to reduce the delamination of the FRP sheet from the RC wall, an anchorage system could be used. Higher quantity of anchors will increase the possibility of FRP rupture due to stress concentrations at the anchors. Therefore, a proper analysis is needed to find the optimal anchorage systems to prevent FRP delamination while minimizing the FRP rupture.

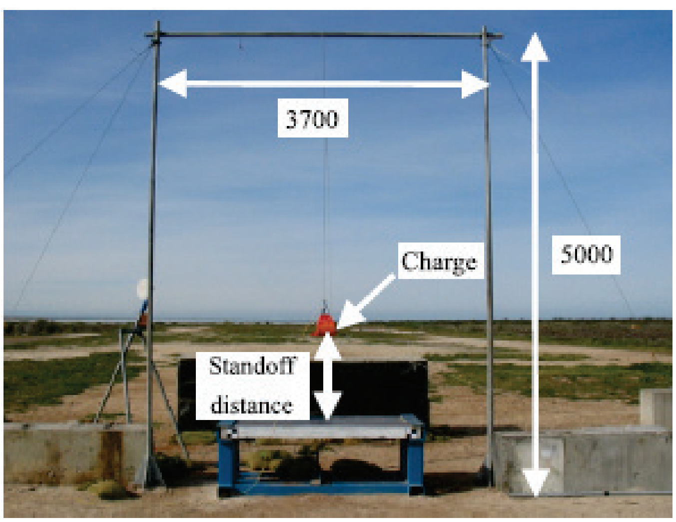
2.5 Slabs

The research that was found for Reinforced Concrete Slabs retrofitted with FRP focused on the performance of the slabs under blast loads. In

situations where bomb blasts occur, the loss of life is primarily a consequence of the collapse of structures and flying concrete debris. Failure of slabs normally does not cause the collapse of a structure, but they can under blast loads be significant contributors for the creation of concrete debris. This has led to research in which concrete slabs were retrofitted to increase their blast load resistance.

Wu et al. (2009) conducted an investigation that focused on the blast resistance of slabs in an effort to mitigate the effects of airblast loads. The study focused on retrofitting a series of reinforced concrete slabs with externally bonded FRP plates and the use of innovative materials such as ultra-high performance fiber concrete (UHPFC) for the construction of new slabs. To perform the investigation, four slabs were constructed with, a normally reinforced concrete (NRC), a reinforced concrete combined with FRP plates, a UHPFC slab without reinforcement, and a UHPFC slab with reinforcement (RUHPFC). The NRC slabs were tested to serve as control specimens. Linear Variable Differential Transformers (LVDTs) were used to record deflection histories, and pressure sensors located at the center and at one edge of the slabs measured airblast pressure time-histories. The explosive charge was centered over the slab using four string guides as shown in Figure 2.7.

Figure 2.7. Charge support frame (dimensions in mm) (Wu et al. 2009).



Evaluation of pressure time-histories showed that the use of end-detonated cylindrical charges will produce shock fronts in the near-field that differ substantially from those assumed in the Navy and Air Force technical manual TM5-1300 (1990) for standard blast design. It was

suggested that for testing structural components, centrally detonated spheres of explosives should be used to simplify the flow field calculations as well as the interpretation of test results. Blast testing indicated that the plain UHPFC slab had less damage than the NRC slabs when subjected to similar blast loads, which confirms that UHPFC is a more effective material for blast design. The performance of the RUHPFC slab was superior to all of slab types tested in the program. Externally bonded FRP strips at the compressive face of the NRC slab improved blast resistance, but the improvement percentage could not be quantified because the slabs were not tested to the point failure initiation.

Tanapornraweekit et al. (2011) published an article that focused on the efficiency of three FRP strengthening schemes for improving the blast performance of RC slabs undergoing single, double, and triple independent explosion tests. One of the schemes was the application of FRP on a single side of the slab. The second scheme involved the application a single layer of FRP on each side of the slabs, forming a FRP sandwich. The third scheme consisted of a double-layer FRP sandwich. All the tested slabs were cast with the same dimensions and the same layout of steel reinforcement.

The test slabs were supported in one direction by a steel test rig in which the trinitrotoluene (TNT) charge was suspended at a distance of 0.50 m (1.6 ft) above the test specimen. In the test program, two control slabs without FRP strengthening and the single-sided FRP slab were subjected to only a single blast test. The single layer FRP sandwich RC slab was subjected to a double blast test, while the double layer FRP sandwich RC slab was subjected to a triple blast test.

The single-sided FRP strengthening scheme prevented the concrete spalling observed in the bare RC slab subjected to the same level of explosion. However, this type of strengthening was not as effective as that offered by the single-layer FRP sandwich strengthening scheme. The FRP sandwich RC slabs were still intact after the second explosion. When the triple explosion test was performed on the double-layered FRP sandwich RC slab, it was observed that under the first two explosions, the test slab performed very well based on real-time monitoring of the shot. The damage to the test slab was very light, and a small number of cracks on the test slab were observed. The preexisting diagonal shear crack initiated during the second explosion led to shear failure and subsequent FRP delamination following the third, highest level of explosion. In addition,

concrete spalling was observed after the third explosion. Although the test slab with two layers of FRP sandwich failed in a brittle manner after the third explosion, the bare RC slab was observed to fail from concrete spalling under a smaller level of explosion. In conclusion, both the one- and two-layer FRP sandwich strengthening schemes were very effective in improving the ductility of the test slabs and enabled the test slab to survive the blast effects from a subsequent explosion.

2.6 Reinforced concrete structures and frames

An extended variety of studies were conducted on reinforced concrete elements such as beams, columns, and beam-column joints. Research was also focused on studying the overall behavior of frame systems and structures that have elements strengthened with FRP. This section presents different studies where a complete structure or frame system was studied. The studies presented in this section describe structures that are strengthened to properly resist either seismic or blast loads.

2.6.1 Seismic retrofit

Elements of a structure are commonly strengthened or retrofitted with FRP to comply with seismic design requirements. But in a real structure, there are different factors that influence structural behavior under seismic loads. Therefore, research has also been focused on the behavior of overall structures that were to some degree retrofitted with FRP.

With the purpose of studying how a structure retrofitted with FRP behaves, Ludovico et al. (2008) made a series of bidirectional pseudodynamic tests on a full-scale three-story reinforced concrete frame structure retrofitted with FRP. The structure was designed only for gravity loads without specific provisions for earthquake resistance. As part of the experiment, the floor plan contained irregularities to create a torsionally unbalanced structure. The floor plan is shown in Figure 2.8. A first test was conducted with a scale Peak Ground Acceleration (PGA) level of 0.20 g. This first test represented the behavior of a typical building configuration. After that, the structure was retrofitted by using FRP laminates. The retrofit design was performed according to the provisions of the guideline developed by Italian National Research Council, CNR-DT 200 (CNR-DT 2004). The columns, beams, and beam-column joints were wrapped with FRP. As can be seen in Figure 2.9, the FRP wrap on the columns extended 31.5 in. from the joints to the columns, and the beam was wrapped through the length where the plastic

hinge was expected to form. The retrofitted structure was then tested under the same input ground motion to have a direct comparison with the previously executed experiment. Finally, in order to investigate the effectiveness of the retrofit technique adopted in the study, another test was conducted with a PGA level of 0.30 g.

Figure 2.8. (a) Plan view and (b) 3-D view of the structure used by Ludovico et al. (2008).

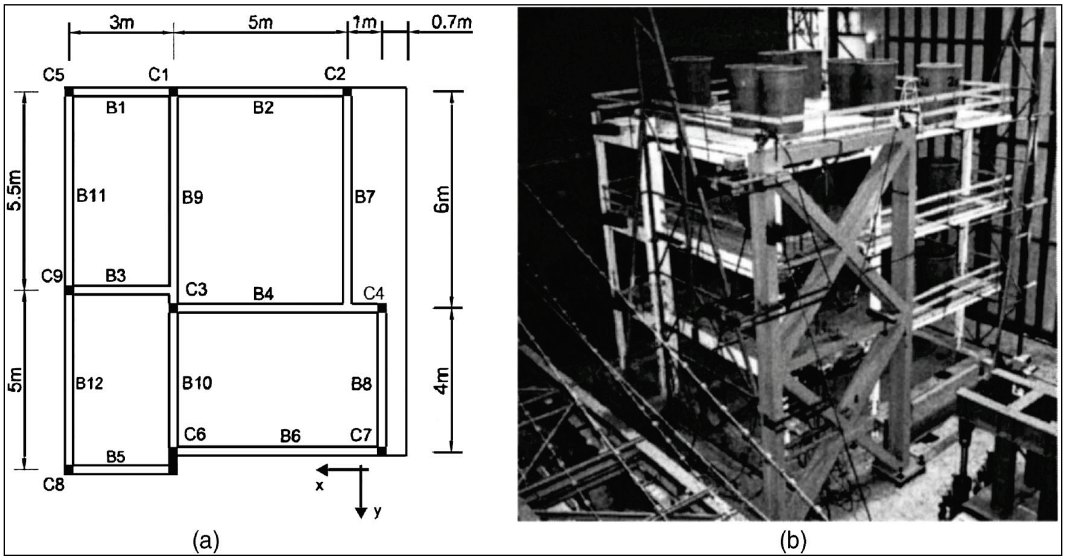
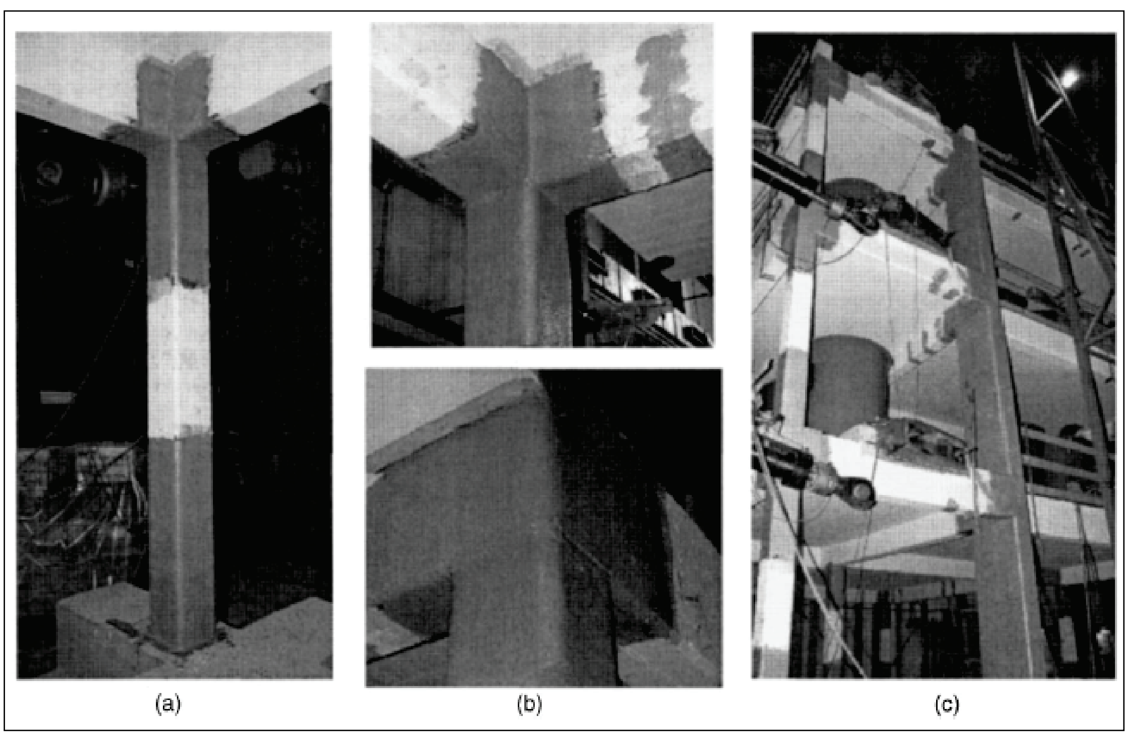


Figure 2.9. GFRP retrofit of the structure: (a) column confinement; (b) unconfined joint retrofit; and (c) overview of the structure after the retrofit (Ludovico et al. 2008).



The experimental results highlighted that the retrofitting procedure provided the structure with an enhanced ductility of approximately 9 percent with respect to the “as-built” configuration. The enhanced ductility implied higher demand on the connections, which could provoke brittle shear failure. According to the authors, the laboratory work showed that the use of composites allowed considerable improvement in the seismic performance of an existing RC structure by increasing its deformation capacity without affecting significantly its stiffness. This indicates that the seismic retrofit design does not require the computation of greater seismic excitations but is mainly governed by the displacement demand.

Pampanin et al. (2007) also studied a seismic retrofit procedure on a frame system with a design based on preseismic code provisions. A multilevel retrofit strategy for upgrading the structure was adopted. The focus of the retrofitting strategy was to make the structure exhibit the desired weak-beam strong-column behavior that is the basis of the design of new seismic resistant RC frames. In the exterior joints, a full retrofit was adopted that protected the column joint and the plastic hinge in the beam. The interior joints were partially retrofitted and protected the column joint while some damage in the beam joint was accepted. Using capacity-demand curves within M-N (moment-axial load) performance domains, the expected sequence of events within the beam-column system were visualized. The retrofitting intervention was experimentally validated by quasistatic cyclic tests on a three-story, three-bay, 2/3 scaled, frame system retrofitted with CFRP sheets. The frame system was subjected to quasistatic cyclic loadings at increasing levels of floor displacements and applied to the structure using three electromechanical actuators connected to the closest beam through a steel extension arm. The presence of gravity loads, which represent a significant portion of the overall capacity in existing underdesigned or gravity-load dominated frames, was simulated using concrete blocks.

After the tests were conducted, the global behavior of the retrofitted frame systems followed the expected and analytical predictions. The frame response was characterized by the formation of plastic hinges in the exterior beams with no damage in the beam-column joints that were protected by the FRP. In the exterior joints, the occurrence of a brittle joint shear mechanism was adequately prevented. This indicates that a more desirable hierarchy of strengths and sequence of events was achieved leading to a more ductile and dissipating hysteresis behavior. In the

interior joints, a controlled minor cracking in the joint panel zone was permitted to protect a column sway mechanism. At a global level, the implementation of the retrofit strategy favored the development of a more appropriate global inelastic mechanism, preventing brittle failure in exterior joints or undesired events such as a soft story mechanism. The authors finalized their work by mentioning issues of accessibility of the joint region that will have to be faced in real applications.

Khaled and El-Sokkary (2008) also studied the behavior of structures designed according to pre-1970 strength-based codes. Using the computer software CANNY, they made an analytical evaluation of three reinforced concrete frame structures before and after being retrofitted with FRP. The structures had5, 10, and 15 stories to represent low, medium, and high-rise buildings. The three buildings had the same floor plan that consisted of three symmetrical bays in both directions. Four different rehabilitation patterns were investigated. Two of the patterns included rehabilitation of both columns and beams. The difference between the patterns was that the rehabilitation in one was made along the full height of the structure, and the rehabilitation in the other was made only in the lower half of the structure height. The other two studied patterns consisted of the rehabilitation of the columns only, where one was along the full height of the structure, and the other was in the lower half only. Nine earthquake records representing a sample of earthquakes with low-, medium-, and high-frequency contents were applied to the three structures.

After analyzing the results, it was concluded that for the low-rise RC frame structure, FRP rehabilitation of the columns only along the full height of the structure was effective in increasing the structure’s Peak Ground Acceleration and maximum interstory drift capacities. On the other hand, FRP rehabilitation of all the columns and beams in the structure did not result in a significant increase in the seismic performance of the rehabilitated structure when compared to the case of rehabilitating the columns only. For the medium and high-rise structures, the rehabilitation of columns only was not as effective as rehabilitation of both columns and beams for the full height of the structure. The FRP rehabilitation of the lower half of the structure was found to be inefficient in all of the three studied buildings. The authors clarified that the drawn results were for the studied cases and the selected earthquakes. In order to generalize the conclusions, further analysis including more earthquake records was necessary.

Another analytical study of existing RC frame buildings and the potential improvements achievable after FRP retrofitting was made by Mortezaeiet al. (2009). Recordings from recent earthquakes provided evidence that ground motions in the near field of a rupturing fault differ from ordinary ground motions, as they can contain a large energy, or "directivity," pulse. Failures of modern engineered structures observed within the near-fault region in recent earthquakes revealed the vulnerability of existing RC buildings against pulse-type ground motions. This may be due to the fact that these modern structures were designed primarily using the design spectra of available standards, which characterize more distant ground motions. Many recently designed and constructed buildings may therefore require strengthening in order to perform well when subjected to near-fault ground motions. To study the behavior of typical existing structures to this type of ground motion and the viability of strengthening them with FRP, the response to near-fault ground motions of six existing reinforced concrete buildings of 3, 6, 10, 14, 16, and 19 stories were evaluated. Buildings with three, six, and 10 stories had a moment-resisting frame system, and the buildings with 14, 16, and 19 stories had a wall-frame system.

The structures were loaded by earthquake loads until a clear plastic collapse mechanism was developed. The main purpose of the retrofitting system design was to force the plastic hinges to form in the beams, changing the failure mode from a column-type to a beam-type collapse mechanism. FRP was used for the rehabilitated joints. For the buildings with a wall-frame system, FRP sheets were used to retrofit the shear walls.

The results showed that for near-fault records, the demands for interstory drifts in the nonretrofitted buildings were different from those for far-fault motions. When the retrofitted structures were subjected to the near-fault records, it was found that the maximum interstory drift value decreased with the FRP strengthening, which led to a more ductile behavior. The energy dissipation of the rehabilitated buildings was 2.3 times that of the original buildings. The analytical results led them to the conclusion that FRP could also be used to strengthen structures subjected to near-fault ground motions, similar to structures subjected to far-fault records.

Niroomandi et al. (2010) developed an analytical evaluation with the main objective of investigating the effects on the seismic performance level and the seismic behavior factor (R) of ordinary RC frames when the joints are

strengthened with FRP sheets. The retrofitting scheme considered in this study was the CFRP web-bonding of the frame joints. A2-D, eight story, three bay, existing RC moment resisting frame was modeled to perform the analysis. The retrofitted joint stiffness in the form of moment-rotation relations was first determined in detailed finite element modeling using the software ANSYS and verified against experimental data. The results of this finite element model were used to conduct nonlinear, static, and pushover analyses of the frame using the software SAP 2000. The web-bonded FRP retrofit at joints resulted in a 40 percent increase in the lateral load resisting capacity of the original RC frame and also enhanced its over-strength by 66 percent. FRP retrofitting of the joints also significantly increased the ductility of the frame, upgrading the ordinary RC frame to an intermediate ductility and even to a special frame. Due to the increased strength capacity and improved ductility, the seismic behavior factor of the frame was substantially increased by over 100 percent, resulting in a more than 50 percent reduction in the seismic base shear.

2.6.2 Retrofitting for blast loads

Although structures can suffer a great level of damage under earthquake loads, they are also vulnerable to large dynamic loads from blasts. These loads can cause the collapse of the entire structure and result in a large number of casualties. Such loads can also lead to the disintegration of walls, creating secondary fragments that will create fatalities in the adjacent rooms.

Malvar et al. (2007) published a paper that reviews the use of composites for retrofitting key structural components such as columns, beams, and walls. The review discusses extensively the retrofitting of reinforced concrete columns, as their failure can cause the collapse of the structure. The use of composite wraps, or steel jackets, could increase the column’s shear capacity, preventing column failure and building collapse. This indicates that retrofit techniques used to resist seismic loads could be extended to resist blast loads, like using FRP wraps to strengthen circular, square, and rectangular columns.

Although retrofitting columns with composite wraps is reminiscent of seismic retrofits, there are many basic differences. The design procedure should address the highly transient dynamic loading that results in apparent material strengthening at high strain rates. For concrete, this phenomenon has been linked to moisture content at lower strain rates

(below about 1/s) and to inertia effects above that. This is typically neglected during seismic loading. Also, the deformed shape of first floor building columns under blast loads usually includes two inflexion points versus a single one for seismic loading. This is an important difference, as the deformed shape with two inflexion points allows for the development of compression membrane response, which can significantly increase the lateral resistance of the column, typically anywhere between 20 percent and 100 percent, depending on the column's geometry and the rigidity of the boundary conditions. To insure ductile flexural behavior, compression membrane requires increased shear capacity, which can be provided by composite hoop wraps (or steel jackets, or, for new construction, additional shear steel in the column, via additional ties or a more closely spaced spiral). Ductile flexural response is necessary to allow the column to dissipate large amounts of energy while still carrying the vertical load.

Detailing is also different in the case of blast loading. For example, in seismic design, a gap is often provided at the top of the retrofitted scheme to prevent increasing the stiffness of the column. This is done to insure that all columns participate equally in resisting the lateral load, otherwise stiffer columns would carry higher loads and fail first. In blast design, typically a single component will be loaded (or loaded significantly more than the others), and its resistance has to be maximized. Hence, in the case of a column, it is preferable to provide no gap at the top (or the bottom) of the retrofit, and instead provide continuity, if possible. For blast design in seismic areas, meeting both requirements, i.e., preventing column stiffness increase while leaving no gap, may require the use of only hoop FRP reinforcement.

Malvar et al. (2007) stated that there is limited research into blast effects on beams. The reason for this is that although columns and load-bearing walls are critical structural elements whose failure could trigger progressive collapse, the failure of a beam usually would only result in localized failure. Beams are also often adjacent to slabs that will provide them significant lateral support, and would therefore be more difficult to fail. Leading from this, the review proceeds to discuss walls and slabs.

Bearing walls are a key structural component whose failure can result in building collapse. Also, the failure of nonbearing walls can result in deadly secondary fragmentation and significant casualties as well. Composites have been used to strengthen concrete and masonry walls from out-of-

plane blast loads. This is different from seismic applications where reinforced walls are often used as shear walls and therefore subjected to in-plane loads.

In columns, the vertical reinforcement tends to rupture at lower lateral displacements. In walls, the composites tend to rupture where cracks form, and although the composites do provide enhanced capacity, the wall ductility and energy dissipation can be limited by the composite's strain to failure. Additionally, the composites can debond at the ends and not enhance the shear capacity along the wall's perimeter, thus requiring separate end anchorage and perimeter shear enhancement mechanisms.

As building construction often utilizes reinforced or unreinforced masonry, Malvar et al. also reviewed the existing work in this area to enable overall conclusions. Similar to the papers that are reviewed in Section 3, retrofitting walls with FRP will increase their blast resistance, even more so if end anchorages are provided to prevent edge debonding. It is noted that while the FRP will increase the flexural capacity, the shear capacity along the wall perimeter may be exceeded and will need to be enhanced independently. In addition, the limited strain capacity of fibers in an epoxy matrix will also limit their energy absorption due to fiber cracking at locations of stress concentration. The reviewed studies by Malvar et al. also showed that the retrofits were successful in containing secondary debris, i.e., debris from the wall itself.

3 Masonry Walls

Masonry walls can be strengthened with FRP to improve their behavior under seismic or blast loads. When the seismic behavior of a wall needs to be improved, the in-plane properties of the wall are the relevant ones since it is in this direction that they will resist the seismic loads. On the other hand, when a wall needs to be improved to resist blast loads, their out-of-plane behavior is the main aspect of study.

El-Dakhakhni et al. (2004) studied the retrofitting unreinforced concrete masonry-infilled steel frame structures using GFRP laminates. The study focused on enhancing the in-plane seismic behavior of unreinforced masonry (URM) infill walls when subjected to displacement controlled cyclic loadings. Six full-scale single-story single-bay steel frames with different infill configurations were tested. One frame was tested without infill in the wall, two other specimens were tested without GFRP laminates, one had a solid wall, and the other had a wall with a symmetrical door opening. The remaining three specimens had their walls retrofitted with GFRP. Two of the retrofitted specimens had GFRP applied at both sides, one being a solid wall and the other had a symmetrical door opening. The last specimen was a solid wall retrofitted on one face only. The frames used for the study had approximate dimensions of 12-ft high and a bay of 10 ft. The nominal standard hollow concrete masonry blocks were certified to meet the provisions of ASTM standards. The mortar used for the construction of the walls was a C-90 standard Type S. The GFRP laminates were cut to the exact dimensions of the wall without adding the laminate to the steel members. Two layers of FRP were applied in order to orient the fibers along both directions of the wall plane. After set arrangement, lateral load was applied by means of a hydraulic actuator to simulate an earthquake load.

The results of this study showed that all the retrofitted specimens had higher stiffnesses and strengths than that of their nonretrofitted counterparts. The study demonstrated URM can be transformed to engineered masonry-GFRP composite walls, in which the GFRP laminates supply the required shear strength while the face shells provide the compressive strength. The GFRP laminates also improve the compressive strength of the face shells. Also, the retrofitting technique maintained the

wall's structural integrity and prevented collapse and debris fallout. The reinforced walls were stable after failure, in contrast to the URM walls. This is particularly convenient in a building because it reduces the seismic hazard associated with the wall falling out of the frame. The GFRP laminates contained and localized the damage of the URM walls even after ultimate failure. The GFRP laminates also made the failure of the wall less catastrophic. The GFRP laminates caused the wall to have a gradual prolonged failure that can dissipate more energy and have major post-peak strength. The GFRP laminates supplied shear strength at the mortar joints. Therefore, they served as external reinforcement for masonry walls that are not properly reinforced for seismic events.

Maji et al. (2008) conducted a study on retrofitted URM walls. Instead of focusing on the seismic behavior of the URM walls, they concentrated their study on the effect of blast loads. They conducted a full-scale blast test on a structure representing a mailroom constructed with unreinforced masonry walls. A rectangular room with unreinforced masonry was built. The room was approximately 20 ft by 12 ft and 13-ft high. The experiment consisted of retrofitting the four walls with various amounts of GFRP layers on the outside face to increase their resistance to blast loads. A charge was located at the center of the room 2.5 ft above the ground. The program BLASTX (WES/SAIC Version 4.1.0 Beta 14, 11 June 1999, MS Windows) developed by the U.S. Army Engineer Waterways Experiment Station, was used to predict blast loadings inside the mailroom test structure. The blast load was produced by the detonation of a 0.91-kg (2-lb) equivalent TNT charge placed near the center of the room. A simple single-degree-of-freedom (SDOF) analytical model was used to predict the structural response of a wall.

The results of the full-scale blast test demonstrated that an optimally designed GFRP retrofit can effectively contain a blast load of known magnitude. The BLASTX code provided the blast pressure time-histories applied to the walls. The SDOF analytical model predicted the structural response of the wall with reasonable accuracy. It was also shown that the approximation of the blast load by exponential and linear decay histories had little effect in the prediction of maximum displacement response. Based on these results, the authors proposed a methodology to design effective retrofit schemes for any wall using an arbitrary retrofit material against a quantifiable internal or external blast load. The first step in the methodology was to determine the blast pressure loads using codes such

as BLASTX or design charts in order to select the kind of retrofit material and amount (thickness) to be used. The SDOF analytical model determined the maximum displacement under the blast pressure history. With the determined maximum displacement the maximum tensile strain was calculated and determined if it is an allowable level of strain in the retrofit material. If necessary, the thickness of the retrofit was modified, and the process was repeated until the calculated strain and the ultimate tensile strain were approximately equal.

Masonry walls subjected to blast loads were also studied by Urgessa and Maji (2010). They tested eight masonry walls reinforced with two and four layers of carbon fiber and two types of polymer matrices. The walls were then subjected to 0.45-kg pentolite booster blast load, with an equivalent TNT value of 1.4 lb, suspended from the ceiling of a test structure. Eight masonry wall segments, 3.33-ft wide by 10-ft high by 8-in. thick, were used. The walls were constructed inside a 10–in. reinforced concrete containment structure. The walls were retrofitted with unidirectional carbon fibers cut to the wall's height. The matrix used in four of the walls was an inorganic matrix (geopolymer) consisting of a liquid potassium silicate solution and amorphous silica powder. The matrix used in the other four walls was an organic matrix consisting of thixotropic epoxy resin and a 2:1 hardener. Angle irons were used to anchor the CFRP to the floor and roof of the test structures. Displacement gauges were mounted on each wall at mid-height to record the deflection history of the walls. Pressure gauges were mounted to record the incident pressure. This permitted measurement of the pressure-time-history caused by the blast and the resulting displacement during the test. The authors presented an analysis based on a nonlinear single-degree-of-freedom method to predict the peak pressure and displacement values for outlining a retrofit design procedure.

The measured blast wave parameters from the test were shown to be in good agreement with the predicted blast wave parameters. Stiffness and failure strain parameters were developed based on a catenary deflected shape. Peak displacement values obtained from the nonlinear single-degree-of-freedom analysis were in reasonable agreement with the test displacement results. Based on the results of the test and the analysis, a design procedure was proposed that could be used to design the number of retrofitted FRP layers needed for wall retrofits. The first step in the retrofit design procedure is to establish the threat to the structure and the required level of protection. Based on the design threat parameters,

determine the wave parameters. In the study, the authors used existing equations for conventional weapon loading to determine the blast wave parameters. These predicted blast wave parameters were in good agreement with the measured blast wave parameters. The next step after determining the blast wave parameters is to run an analysis using a nonlinear SDOF method. A number of FRP retrofits are assumed, and the nonlinear SDOF model is executed. After determining the peak deflection value from the displacement-time-history response, this value has to be verified that it does not exceed the displacement limit. After revising the number of FRP retrofit layers as necessary, the last step is to detail proper FRP application guidelines and securing mechanisms. Detailed securing mechanisms should be provided to transfer loads from the composite material to the surrounding component of the structure.

Cheng and McComb (2010) investigated the out-of-plane behavior of unreinforced masonry walls (URM) through impact tests using a drop-weight pendulum on full-scale concrete masonry walls externally strengthened with CFRP composites. The wall specimens were constructed from concrete masonry units (CMUs) with a standard size of 8 in. by 8 in. by 16 in. The wall specimens were 4-ft wide, 4-ft high, and 8-in. thick. Three strengthening schemes on one side of the wall were studied, continuous unidirectional and continuous woven sheets, discrete strips in a vertical pattern, and discrete strips in orthogonal and diagonal patterns.

The authors concluded that CFRP strengthening substantially prolonged the cracking and failure of URM walls under pendulum impact. The continuously applied woven CFRP fabric increased the impact resistance of the wall more than the continuously applied unidirectional fabric. The woven fabric discretized into vertical strips was more effective than sheets when at least one strip was directly behind the impact location. With the same amount of FRP strip material, the five-strip wall with narrower, but more closely spaced strips performed slightly better than the three-strip wall. For all walls, typical failure modes were vertical cracks in masonry units occurring between the CFRP strips. No debonding or rupture of CFRP materials was observed. The strengthened walls with an orthogonal pattern of unidirectional strips performed better than the one using a diagonal pattern. Orthogonal and diagonal walls using unidirectional strips withstood more impact trials than the wall using continuous unidirectional sheet. The URM wall strengthened with continuous woven sheets withstood the highest impact loads among all nine walls. Several

other parameters could be included in future tests such as strength of concrete masonry blocks, other types of FRP reinforcement, and different types of impact loads, e.g., projectile motion.

Hamed and Rabinovitch (2011) studied the natural frequencies and out-of-plane vibration modes of one-way masonry walls strengthened with composite materials. Due to the inherent nonlinear behavior of the masonry wall, the dynamic characteristics depend on the level of out-of-plane load (mechanical load or forced out-of-plane deflections) and the resulting cracking, nonlinear behavior of the mortar material, and debonding of the composite system. To account for the nonlinearity and the accumulation of damage, a general nonlinear dynamic model of the strengthened wall was developed. The model was mathematically decomposed into a nonlinear static analysis phase, in which the static response and the corresponding residual mechanical properties were determined, and a free vibration analysis phase, in which the dynamic characteristics were determined.

After conducting the analysis, the authors concluded that opposed to the relatively small impact of the strengthening system on the dynamic properties of RC beams, the strengthening system significantly altered the dynamic characteristics of the masonry wall. In that sense, the dynamic properties of the strengthened wall become less sensitive to the static loading. The results also showed that the differences between the magnitudes of the natural frequencies in successive modes were rather small. As the dynamic or seismic response of the wall may be affected by the higher frequencies, the model had quantified the higher vibration modes and addressed the impact of the strengthening system on the entire spectrum of natural frequencies, on the spacing between successive modes, and on the rate of deterioration of the entire spectrum due to accumulation of damage.

4 Steel

Most of the successful applications of FRP reported in the literature are for strengthening structural elements made of concrete, masonry, and wood. There is a vast variety of applications where FRP is used to strengthening concrete members. Numerous applications have been successfully found to provide additional capacity to beams, columns, and beam-column joints. Even research on full-scale structures retrofitted with FRP can be found in the literature. Research work has also focused on improving the behavior of concrete walls and masonry walls. In contrast, the amount of research work available that is related to the strengthening with FRP of steel elements and structures is considerably limited. According to Okeil et al. (2009), this is because steel has superior mechanical properties (yield strength and elastic modulus) when compared to concrete; hence, the effectiveness of using commonly used FRPs is greatly reduced since larger amounts of FRP would be needed. The availability of high-modulus FRP materials in recent years has showed potential for new steel strengthening applications. The research found in the literature still focuses in the improvement of mechanical properties of elements without considering the effects that dynamic loads may create.

4.1 Beams and Girders

Okeil et al. (2009) studied the strengthening of steel structures by introducing additional stiffness to buckling-prone regions. Their study considered two types of specimens, i.e., bars were used to study specimens in tension, and beams were used to study specimens in shear. The purpose of the bar (tension) specimen is to provide a benefit comparison between the commonly used strengthening approach and a new proposed technique. The commonly used strengthening approach relies on in-plane FRP contribution to the behavior of the strengthened member, while the proposed method enhances its strength by relying on the out-of-plane stiffness of the geometric properties of pultruded FRP sections. These pultruded FRP sections were used as stiffeners in the beam. Web buckling was chosen as the investigated mode of failure of the steel beam. To force the beam to fail from web buckling, all other possible modes of failure (flexure, local flange buckling, steel stiffener buckling, welding, and load bearing) were prevented by overdesigning the non-strengthened section by a factor of at least 2.0. The GFRP stiffeners were cut from a pultruded

wide-flange beam to form a T-shape section. One GFRP stiffener was bonded to each side of the steel web in a vertical orientation. The area of the stiffener flange was the only bonded surface between the steel web and the GFRP stiffener. A single point load was applied over the first internal stiffener on one side of the beam causing the first panel to be subjected to three times the shear force acting on the rest of the beam.

The GFRP section was bonded to thin-walled steel plates in an orientation that contributes mainly to the out-of-plane stiffness of the plate rather than the in-plane strength. Because of the GFRP stiffener's orientation, it was possible to use low-modulus FRP materials rather than the more expensive high-modulus materials that are usually used to strengthened steel structures. Two types of specimens were tested to demonstrate the difference in behavior between both strengthening techniques (in-plane and out-of-plane). The tension bar (specimen strengthened in-plane) was found to have an increase in stiffness of 13 percent. In comparison, a 56 percent higher load was needed to cause the stiffened beam specimen (out-of-plane strengthening technique) to fail. The existing code was used to estimate the increase in shear capacity of steel beams with FRP stiffeners, assuming that the GFRP stiffener behaved in an identical manner as steel stiffeners. Based on this assumption, the code estimates a 219 percent increase in strength. This estimate was substantially higher because the debonding of the GFRP stiffener was not accounted for. Thus, it can be stated that new formulas will need to be developed to estimate the strength of GFRP-stiffened steel webs, which will require a concerted research effort to cover the various parameters that may affect the performance of the proposed technique.

Hmidan et al. (2011) presented an experimental program to study the behavior of notched steel beams repaired with carbon-fiber-reinforced polymer (CFRP) sheets. Particular attention was paid to examining the interaction between the level of initial damage (i.e., notch depth) and CFRP repair. Multiple stages of fatigue-crack-propagation in a steel beam were simulated by various notch sizes, including ratios a_o/h = 0.1, 0.3, and 0.5, in which a_o is the notch depth and h is the beam height. The experiment was assessed using W4 × 13 A992 hot-rolled steel sections. CFRP was bonded to the tensile soffit of the notched beams with an epoxy adhesive. All beams were simply supported and loaded in four-point bending. A 3-D FE model was developed to predict the behavior of the experimental beams using the general-purpose FEA program ANSYS. The

models accounted for the effect of crack propagation and the debonding process of the CFRP.

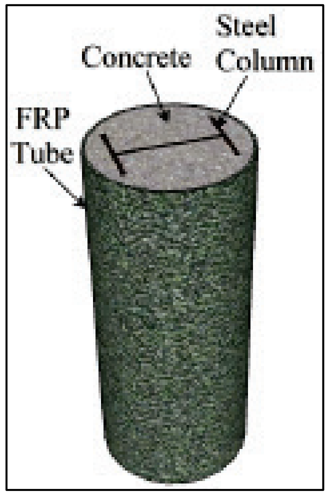
The authors found that the load capacity and serviceability of the repaired beams were significantly affected by the level of initial damage (notch size). The efficacy of the CFRP repair was more pronounced when the damage level increased. The load-sharing mechanism between the CFRP and steel substrate improved the yield capacity of the repaired beams. Such an effect, however, was not noticeable when significant debonding of the CFRP took place. CFRP sheet stabilized the Crack Mouth Opening Displacement (CMOD) of the repaired steel beams. The linearity of the CMOD was improved until substantial debonding of the CFRP occurred. The level of initial damage in the beams influenced the postpeak behavior of the CMOD, which was related to the internally stored energy and the crack-tip plasticity of the repaired beams. Failure mode of the repaired beams was independent of the level of initial damage, whereas the damage level influenced the web fracture rate of the repaired beams. The plastic region above the crack tip of the beams was dominated by the damage level. The crack-propagation rate across the repaired steel section approached that of the unrepaired counterpart when significant CFRP debonding occurred. The level of initial damage governed the initiation of CFRP debonding; however, its contribution to the debonding behavior was not significant once the CFRP was completely debonded in the vicinity of the damage location. The need for an improved bond-slip model was addressed to better predict the debonding progression of the CFRP bonded to a steel beam.

4.2 Columns

Karimi et al. (2011) investigated the effect of the slenderness ratio on the behavior of a FRP-encased steel-concrete composite column. As seen in Figure 4.1, the composite column specimens were constructed by placing an FRP tube around a steel column, and the void between the steel section and the FRP tube was subsequently filled with concrete. The proposed composite system may also be applied to retrofitting existing steel columns to increase the load carrying capacity, axial stiffness, and energy dissipation capacity. Nine column specimens were tested in the experimental program, six were composite column specimens and the remaining three were bare steel column control specimens tested for comparative purposes. The bare steel column specimens were composed of short, intermediate length, and slender columns varying between 20 in. and 120 in. in height with

slenderness ratios from 0.3 to 1.6, respectively. The composite columns height was also in the range of 20 in. to 120 in. The slenderness ratio of the corresponding composite column specimens was between 0.1 and 0.9.

Figure 4.1. Proposed composite system by Karimiet al. (2011).



After applying a constant increment of axial load, the authors concluded that the FRP tube provided significant confinement to the concrete in the short composite column specimens, which resulted in an 80 percent increase in the compressive strength of the concrete. Composite column specimens with a slenderness ratio of greater than 0.6 failed at small axial strains due to loss of stability prior to efficient confinement activation. The ratio of the compressive strength, elastic axial stiffness, and energy dissipation of capacity of the composite column specimens to those of the steel column specimens were 5-10, 4-6 and 15-25, respectively. Failure of the short steel column specimen having a slenderness ratio of 0.3 (20 in. in height) occurred due to yielding of the entire cross section followed by local buckling of the steel flanges and web. Failure of the short composite column specimen with a slenderness ratio of 0.1 (20 in. in height) occurred due to the rupture of the FRP tube under lateral tension followed by crushing and spalling of the concrete. The steel and composite column specimens with a slenderness ratio of greater than 0.3 and 0.1, respectively, failed by overall buckling due to the loss of stability.

The proposed composite system significantly reduced the slenderness ratio of long steel column specimens by providing stability against overall

buckling, resulting in enhanced compressive strength in the composite column specimens.

Shaat and Fam (2009) conducted an experimental investigation focusing in the behavior of slender steel columns strengthened using high-modulus (45,000 ksi) carbon fiber-reinforced polymer (CFRP) plates. A total of 18 steel columns, cut from 44 by 44 by 3.2 mm (1.75 by 1.75 by 1/8 in.) square hollow structural section (HSS) members were tested in compression under concentric loading. The specimens consisted of six sets in triplicates. Three sets served as control nonstrengthened columns with slenderness ratios of 46, 70, and 93. The other three sets were the CFRP-strengthened counterparts. Unidirectional pultruded CFRP plates were used and cut in order to provide the desired CFRP reinforcement ratio. All column specimens were tested under concentric loading. Lubricated cylindrical bearings were used at both ends of the specimen to allow for free end rotation in one plane only. A simplified analytical model was proposed to predict the ultimate axial load of FRP-strengthened slender steel columns, based on the ANSI/AISC 360-05 provisions, which were modified to account for the transformed section properties and a failure criteria of FRP derived from the experimental results.

Through this investigation, it was found that the effectiveness of the CFRP system in increasing the axial strength of the columns was substantial as slenderness ratios (kL/r) become larger. The axial load capacity of HSS steel columns tested in this study was increased by 6 percent, 35 percent, and 71 percent for columns with kL/r of 46, 70, and 93, respectively. The axial stiffness of HSS steel columns was also increased due to application of CFRP; however, the increase was only slightly affected by kL/r. The stiffness of the columns was increased by 10 percent, 16 percent, and 17 percent for columns with kL/r of 46, 70, and 93, respectively. CFRP failure could occur prior to or after overall buckling of the column, depending on the slenderness ratio. At a kL/r of 46, CFRP debonding occurred prior to buckling, while at a kL/r of 93, CFRP crushing occurred after overall buckling. At a kL/r of 70, CFRP debonding and overall buckling occurred almost simultaneously. The contribution of CFRP was incorporated into the provisions of ANSI/AISC 360-05 by using the transformed section properties as well as an average strain failure criterion of CFRP. The modified provisions can be used to predict the axial strength of CFRP-strengthened columns.

The model showed that there could be a critical kL/r value at the low end, below which CFRP plates debond early at both sides and before overall buckling occurs with no gain in strength. There could also be another critical kL/r value at the very high end, beyond which CFRP crushes early at the inner side and before buckling occurs, and the peak load is governed by the capacity when CFRP remains on the outer side only.

Previously to developing their experimental work, Shaat and Fam (2007) developed an analytical model to predict the behavior of concentrically loaded square HSS slender columns, strengthened with high-modulus CFRP sheets. The study included five slender 3.5 by 3.5 by 1/8 in. HSS column specimens with a nominal yield stress and tensile strength of 55 and 70 ksi, respectively. The length of the pin-ended columns was 94 in., giving a slenderness ratio KL/r of 68. The model predicted the load versus axial and lateral displacements and accounted for plasticity of steel, the built-in through-thickness residual stresses, geometric nonlinearity, initial out-of-straightness imperfection, and the contribution of CFRP sheets. The model was verified using test results.

After analyzing the test results, it was concluded that externally bonded longitudinal CFRP sheets were effective in increasing the axial strength and stiffness of slender HSS columns. When five layers of CFRP sheets were applied on two opposite faces of the column, the percentage increased in strength ranged between 11 percent and 39 percent depending on the value of out-of-straightness. Axial stiffness is also increased by up to 46 percent, regardless of the value of out-of straightness. The effectiveness of CFRP retrofitting increased as the values of out-of-straightness of the column increased. However, for a given CFRP reinforcement ratio, there could be a certain level of out-of-straightness beyond which the gain in strength becomes constant. On the other hand, out-of-straightness has a negligible effect on the gain in stiffness. The effectiveness of the CFRP retrofitting increased for columns with higher slenderness ratios that did not exhibit high residual stresses. Ignoring residual stresses, steel plasticity, or crushing of CFRP associated with local buckling could overestimate the axial strength of the column.

Teng and Hu (2007) studied the performance of circular hollow steel tubes. Before presenting their analytical model, they studied the behavior of tubes under compressive axial load. Four cylinders were tested. The first specimen had a bare configuration, the second had a single ply of CFRP,

the third had two plies of CFRP, and the fourth had three plies of CFRP along the full length. The failure mode of the bare steel tube was outward buckling around the circumference. This local buckling mode near the tube end, widely known as the elephant's foot buckling mode (Figure 4.2), is normally found in steel tubes whose diameter-to-thickness ratio is relatively small.

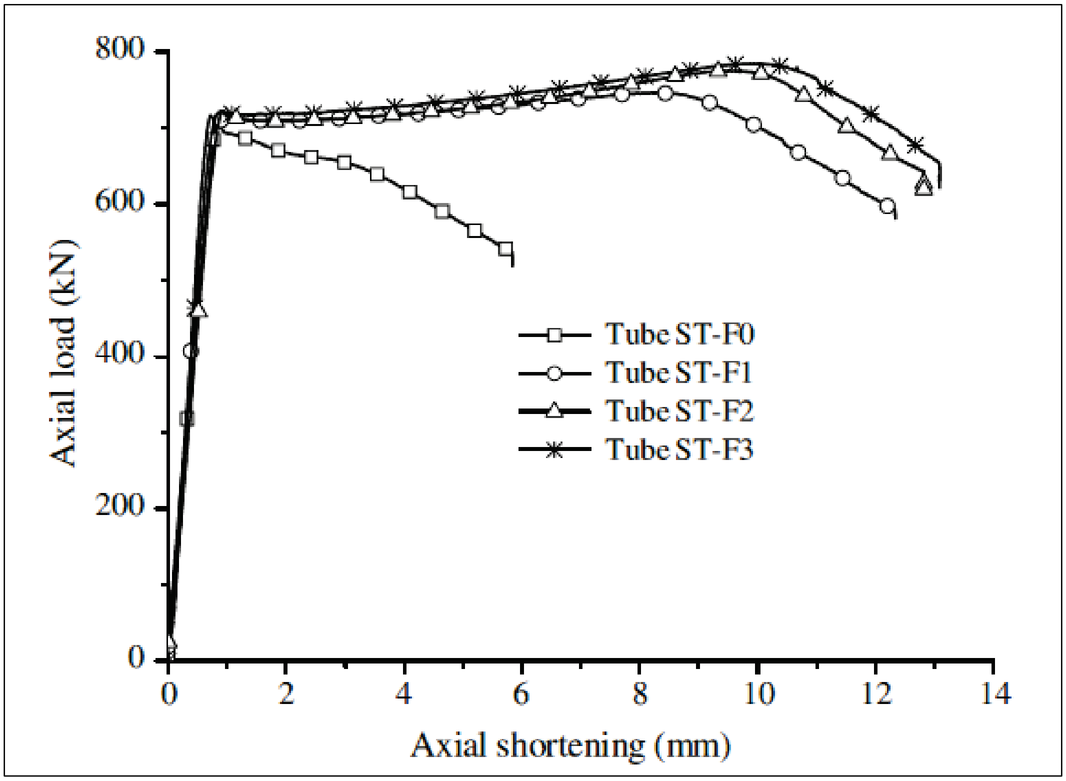
Figure 4.2. Elephant's foot buckling mode (Teng and Hu 2007).

In Figure 4.3, the behavior of the three retrofitted tested specimens is compared with that of the bare tube. The tubes with more plies of CFRP showed more ductility. It is important to say that CFRP did not increase the axial capacity of the section.

In conclusion, the authors found that the failure mode of the bare steel tube was outward buckling around the circumference. The three tubes that were jacketed with the GFRP exhibited a ductile strength curve compared with the bare tube. In the steel tube with a single-ply FRP jacket, failure involved outward local buckling deformations near the ends causing the FRP jacket to eventually rupture due to hoop tension. In the tube with a two-ply FRP jacket, the FRP jacket also ruptured near one of the ends due to the expanding local buckling deformations, but inward buckling deformations became more important in this tube. When a three-ply FRP jacket was used, local rupture of the FRP jacket did not occur, and failure was dominated by inward buckling deformations away from the two ends. FRP confinement of circular hollow steel tubes led to great increase in ductility with very limited increases in strength, a feature that is highly desirable in the seismic retrofit of structures. It was found that the finite

element predictions were sensitive to the chosen imperfection parameters only in the final stage of deformation. The choice of a geometric imperfection for the finite element model of an FRP-confined steel tube with a more rational basis is an issue that requires further investigation. The finite element model showed that at the ultimate load for the steel tube confined with a single-ply FRP jacket, the hoop strains in the jacket at the crest of the elephant's foot buckle are higher than those elsewhere.

Figure 4.3. Behavior of tested circular hollow steel tubes (Teng and Hu 2007).



5 Conclusions

This report presented a review of the state of practice regarding the use of composite materials to retrofit structures and structural elements that are subjected to dynamic loads. Specifically, the study looked into earthquake and blast loads. It was found that the majority of applications and research available for the use of FRP in strengthening structures under dynamic loads focused on reinforced concrete structures and elements. The main applications were for retrofitting reinforced concrete structures that were designed using preseismic building codes or structures that can be exposed to blast loads. In the case of masonry structures and elements, the majority of the research focused on retrofitting masonry walls with FRP to improve their behavior when subjected to blast loads. A limited amount of research was conducted on retrofitting steel structures and structural elements with FRP, and the research being made focused on the improvement of mechanical properties without considering the effects of dynamic loads.

There is a considerable amount of research that has been conducted on retrofitting reinforced concrete elements that were designed before the seismic requirements were introduced. The available research focused on retrofitting frame elements such as columns and beam-column joints since these two elements commonly do not have the required detailing of new code provisions. FRP jackets are a suitable solution for this problem since they provide additional confinement for columns, increasing the compressive strength, reducing the lap splice length, and increasing the shear strength. Different methods and configurations for applying FRP jackets were discussed, all fulfilling their purpose with different levels of effectiveness being achieved. The extent of the research on retrofitting these elements with FRP is highly developed, and studies focusing on the behavior of overall frame structures that include elements retrofitted with FRP have been performed. As expected, the results proved that retrofitting elements with FRP added ductility to the structure, thus increasing the level of energy dissipation. The use of FRP jackets to satisfy actual code requirements is not limited to frame elements. A considerable amount of research has been conducted that studied the use of FRP to retrofit bridge columns and piers that are not properly detailed for seismic events.

The study of blast loads in reinforced concrete structures and elements focused on the study of walls and slabs. Walls suffer substantial damage when subjected to out-of-plane blast loads. This is different from seismic applications in which walls are often used as shear walls and are subjected to in-plane loads. Therefore, it is often necessary to retrofit them using FRP and improve their blast loading resistance capacity. Several researchers concentrated their efforts on this subject, and their investigations proved the effectiveness of using FRP to increase the wall's blast load resistance capacity. The research that focused on reinforced concrete beams subjected to blast loads is limited. The studies reviewed indicate that FRP can potentially improve the behavior of beams under blast loads, but further research is necessary to fully understand the behavior of beams strengthened with FRP.

Similarly to reinforced concrete walls, masonry walls can also be strengthened with FRP. If the in-plane seismic behavior of the wall needs to be improved, research shows that the use of FRP laminates is a suitable solution. Research also shows that masonry walls can be strengthened with FRP to improve their out-of-plane behavior. As seen in different studies, masonry walls can be strengthened with FRP to improve their behavior when subject to blast loads. Something that needs to be considered is that the retrofitting of masonry walls affects the dynamic characteristics of these walls. Research relating these changes to the dynamic characteristics of a masonry wall with the behavior of an overall structure under dynamic loads was not found in the literature.

Work related to the applications on strengthening steel elements with FRP is very limited. The use of FRP to improve the characteristics of steel elements is relatively modern; hence, the research found in the literature still focuses on improving the mechanical properties of steel elements. Studies focusing on the behavior of a steel element subjected to dynamic loads were not found. One study concentrated on the application of pultruded FRP sections as beam stiffeners. The application was validated through experimental static tests on specimens with different retrofitting schemes. A good follow-up of this work is to subject similar specimens to dynamic loads to verify if other failure mechanisms such as debonding occurs.

References

Al-Salloum, Y. A., and T. H. Almusallam. 2007. Seismic response of interior RC beam-column joints upgraded with FRP sheets. I: Experimental study. *Journal of Composites for Construction* 11(6):575-589.

Almusallam, T. H., and Y. A. Al-Salloum. 2007. Seismic response of interior RC beam-column joints upgraded with FRP sheets. II: Analysis and parametric study. *Journal of Composites for Construction* 11(6):590-611.

Al-Salloum, Y. A., N. A. Siddiqui, H. M. Elsanadedy, A. A. Abadel, and M. A. Aqel. 2011. Textile-reinforced mortar versus FRP as strengthening material for seismically deficient RC beam-column joints. *Journal of Composites for Construction* 15(6):920-933.

Alsayed, S. H., T. H. Almusallam, Y. A. Al-Salloum, and N. A. Siddiqui. 2010. "Seismic rehabilitation of corner RC beam-column joints using CFRP composites." *Journal of Composites for Construction* 14(6):681-692.

American Association of State Highway Transportation Officials. 1977. Standard specifications for highway bridges. Washington DC: AASHTO.

American Concrete Institute. 1995. *Building code requirement for structural concrete and commentary*. ACI 318-95. Farmington Hills, MI: American Concrete Institute.

_____. 2002. *Building code requirement for structural concrete and commentary*. ACI 318-02. Farmington Hills, MI: American Concrete Institute.

_____. 2007. *Report on fiber-reinforced polymer (FRP) reinforcement for concrete structures*. ACI 440R-07. ACI Committee 440. Detroit: American Concrete Institute.

_____. 2008. *Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures*. ACI 440.2R-08, ACI Committee 440, Detroit: American Concrete Institute.

Baghiee, N., M. R. Esfahani, and K. Moslem. 2009. Studies on damage and FRP strengthening of reinforced concrete beams by vibration monitoring. *Engineering Structures* 31:875-893.

Bakis, C. E., L. C. Bank, V. L. Brown, E. Cosenza, J. F. Davalos, J. J. Lesko, A. Machida, S. H. Rizkalla, and T. C. Triantafillou. 2002. Fiber-reinforced polymer composites for construction—State-of-the-art review. *Journal of Composites for Construction* 6(2):73-87.

Barbero, E. J., J. F. Davalos, S. A. Kiger, and J. S. Shore. 1997. Reinforcement with advanced composite materials for blast loads. *Proceedings of Structures Congress XV*, The Society, New York, 663-667.

Breña, S. F., and B. M. Schlick. 2007. Hysteretic behavior of bridge columns with FRP-jacketed lap splices designed for moderate ductility enhancement. *Journal of Composites for Construction* 11(6): 565-574.

Cheng, L., and A. McComb. 2010. "Unreinforced concrete masonry walls strengthened with CFRP sheets and strips under pendulum impact." *Journal of Composites for Construction* 14(6):775-783.

Chu, K. D., T. J Lu, and Y. M. Lin, 1998. Rep., Industrial Technology and Research Institute, Taiwan, 148–156.

CNR-DT 200. 2004. Instructions for design, execution and control of strengthening interventions by means of fibre-reinforced composites. Italian National Research Council (2004).

El-Dakhakhni, W. W., A. A. Hamid, and M. Elgaaly. 2004. Seismic retrofit of concrete-masonry-infilled steel frames with glass fiber-reinforced polymer laminates. *Journal of Structural Engineering* 130(9).

ElSouri, A. and M. H. Harajli. 2011. Seismic repair and strengthening of lap splices in RC columns: Carbon fiber–reinforced polymer versus steel confinement. *Journal of Composites for Construction* 15(5):721-731.

Faza, S., H. GangaRao, and E. J. Barbero. 1994. Fiber composite wrap for rehabilitation of concrete structures. *Repair and Rehabilitation of the Americas*: 181-192.

Fujii, M., Kobayashi, K., Miyagawa, T., Inoue, S., and Matsumoto, T. 1988. A study on the application of a stress-strain relation of confined concrete. *Proc., JCA Cement and Concrete, Japan Cement Association, Tokyo*. 42: 311-314.

Galal, K., and H. El-Sokkary. 2008. Analytical evaluation of seismic performance of RC frames rehabilitated using FRP for increased ductility of members. *Journal of Performance of Constructed Facilities* 22(5):276-288.

Hamed, E., and O. Rabinovitch. 2005. Dynamic behavior of reinforced concrete beams strengthened with composite materials. *Journal of Composites for Construction* 9(5):429-440.

Hamed, E., and O. Rabinovitch. 2011. Free out-of-plane vibrations of masonry walls strengthened with composite materials. *Journal of Engineering Mechanics* 137(2):125-137.

Hmidan, A., Y. J. Kim, and S. Yazdani. 2011. CFRP repair of steel beams with various initial crack configurations. *Journal of Composites for Construction* 15(6):952-962.

Hoshikuma, J., K. Kawashima, K. Nagaya, and A. Taylor. 1997. Stress-strain model for confined reinforced concrete in bridge piers. *J. Struct. Eng. (India)* 123(5):624-633.a68.

Howie, I, and V. M. Karbhari. 1995. Effect of tow sheet composite wrap architecture on strengthening of concrete due to confinement: I - Experimental studies. *Journal of Reinforced Plastics and Composites* 14(9):1008-1030.

Ilki, A., I. Bedirhanoglu, and N. Kumbasar. 2011. Behavior of FRP-retrofitted joints built with plain bars and low-strength concrete. *Journal of Composites for Construction* 15(3):312-326.

Javed, S. 1996. Accelerated aging in concrete beams externally bonded with carbon fiber tow sheet. Problem Report, West Virginia University, Morgantown, WV.

Jerome, D. M. 1996. Dynamic response of concrete beams externally reinforced with fiber reinforced plastic. PhD diss., the University of Florida, Gainsville, FL.

Karimi, K., M. J. Tait, and W. W. El-Dakhakhni. 2011. Influence of slenderness on the behavior of a FRP-encased steel-concrete composite column. *Journal of Composites for Construction*.

Kent, D. C., and R. Park1971. Flexural members with confined concrete. *J. Struct. Div.* 97(7):1969-1990.jtd.

Khaled, G., and H. El-Sokkary. 2008. Analytical evaluation of seismic performance of RC frames rehabilitated using FRP for increased ductility of members. *Journal of Performance of Constructed Facilities* 22(5):276-288.

Kowalsky, Mervyn J., and M. J. Nigel Priestley. 2000. Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions. *ACI Structural Journal* 97.3.

Lehman, Dawn Ellen, and Jack P. Moehle. 2000. Seismic performance of well-confined concrete bridge columns. Pacific Earthquake Engineering Research Center.

Li, B., and Q. Kao. 2011. Seismic behavior of reinforced concrete interior beam-wide column joints repaired using FRP. *Journal of Composites for Construction* 15(3):327-338.

Li, B., and C. L. Lim. 2010. Tests on seismically damaged reinforced concrete structural walls repaired using fiber-reinforced polymers. *Journal of Composites for Construction* 14(5):597-608.

Ludovico, M. D., G. Manfredi, E. Mola, P. Negro, and A. Prota. 2008. Seismic behavior of a full-scale RC structure retrofitted using GFRP laminates. *Journal of Structural Engineering* 134(5):810-821.

Maji, A. K., J. P. Brown, and G. S. Urgessa. 2008. Full-scale testing and analysis for blast-resistant design. *Journal of Aerospace Engineering* 21(4):217-225.

Malvar, L. J., J. E. Crawford, and K. B. Morrill. 2007. Use of composites to resist blast. *Journal of Composites for Construction* 11(6):601-610.

Mander, J. B., M. J. Priestley, and R. Park. 1988. Theoretical stress-strain model for confined concrete. *J. Struct. Eng.* 114(8):1804-1826.a3p [Abstract].

Meier, U., and A. Winistorfer. 1995. Retrofitting of structures through external bonding of CFRP sheets. *Proceedings of the Second RILEM Symposium*, RILEM Publications, France, 23-25.

Morrill, K. B., L. J. Malvar, and J. E. Crawford. 2004. Blast resistant design and retrofit of reinforced concrete columns and walls. *Proceedings of structures congress-building on the past-securing the future*.

Mortezaei, A., H. R. Ronagh, and A. Kheyroddin. 2009. Seismic evaluation of FRP strengthened RC buildings subjected to near-fault ground motions having fling step. *Composite Structures* 92:1200-1211.

Muguruma, H., S. Watanabe, S. Katsuta, and S. Tanaka. 1980. A stress-strain model of confined concrete. In *Proceedings, JCA Cement and Concrete* 34:429-432. Japan Cement Association.

Mutalib, A. A., and H. Hao. 2011. Numerical analysis of FRP-composite-strengthened RC panels with anchorages against blast loads. *Journal of Performance of Constructed Facilities* 25(5):360-372.

Niroomandi, A., A. Maheri, M. R. Maheri, and S. S. Mahini. 2010. Seismic performance of ordinary RC frames retrofitted at joints by FRP sheets. *Engineering Structures* 32:2326-2336.

Okeil, A. M., Y. Bingol, and R. Ferdous. 2009. Novel technique for inhibiting buckling of thin-walled steel structures using pultruded glass FRP sections. *Journal of Composites for Construction* 13(6):547-557.

Pampanin, S., D. Bolognini, and A. Pavese. 2007. Performance-based seismic retrofit strategy for existing reinforced concrete frame systems using fiber-reinforced polymer composites. *Journal of Composites for Construction* 11(2):211-226.

Perrone, M., J. A. O. Barros, and A. Aprile. 2007. CFRP-based strengthening technique to increase the flexural and energy dissipation capacities of RC columns. *Journal of Composites for Construction* 13(5): 372–383.

Qi, Xiaoxuan, and Jack P. Moehle. 1991. Displacement design approach for reinforced concrete structures subjected to earthquakes. *Earthquake Engineering Research Center 91(2)*, College of Engineering/University of California.

Realfonzo, R., and A. Napoli. 2009. Cyclic behavior of RC columns strengthened by FRP and steel devices. *Journal of Structural Engineering* 135(10):1164-1176.

Ross, C. A., M. R. Purcell, and E. L. Jerome. 1997. Blast response of concrete beams and slabs externally reinforced with fiber reinforced plastics (FRP). *Proceedings of Structures Congress XV*, The Society, New York, 673-677.

Saatcioglu, M., and S. R. Razvi. 1992. Strength and ductility of confined concrete. J. *Struct. Eng. (India)* 118(6):1590-1607.a68.

Saiidi, S. M., F. Martinovic, B. McElhaney, D. Sanders, and F. Gordaninejad. 2003. Assessment of steel and fiber reinforced plastic jackets for seismic retrofit of reinforced concrete columns with structural flares. *ASCE Journal of Structural Engineering* 130(4):609-617.

Scott. D., and P. Mlakar. 1998. Dynamic response of concrete members externally reinforced with carbon fiber-reinforced polymeric materials. *Proceedings of the 69th Shock and Vibration Symposium*, Minneapolis.

Shaat, A., and A. Fam. (2007). Fiber-element model for slender HSS columns retrofitted with bonded high-modulus composites. *Journal of Structural Engineering* 133(1):85-95.

Shaat, A., and A. Fam. 2009. Slender steel columns strengthened using high-modulus CFRP plates for buckling control. *Journal of Composites for Construction* 13(1):2-12.

Shan, B., Y. Xiao, and Y. Guo. 2006. Residual performance of FRP-retrofitted RC columns after being subjected to cyclic loading damage. *Journal of Composites for Construction* 10(4):304-312.

Sheikh, Shamim A., and S. M. Uzumeri. 1982. Analytical model for concrete confinement in tied columns. *Journal of the Structural Division* 108.12(1982): 2703-2722.

Tanapornraweekit, G., N. Haritos, and P. Mendis. 2011. Behavior of FRP-RC slabs under multiple independent air blasts. *Journal of Performance of Constructed Facilities* 25(5):433-440.

Teng, J. G., and Y. M. Hu. 2007. Behaviour of FRP-jacketed circular steel tubes and cylindrical shells under axial compression. *Construction and Building Materials* 21:827-838.

Urgessa, G. S., and A. K. Maji. 2010. Dynamic response of retrofitted masonry walls for blast loading. *Journal of Engineering Mechanics* 136(7):858-864.

Wu, Y.-F., T. Liu, and L. Wang. 2008. Experimental investigation on seismic retrofitting of square RC columns by carbon FRP sheet confinement combined with transverse short glass FRP bars in bored holes. *Journal of Composites for Construction* 12(1):53-60.

Wu, C., D. J. Oehlers, J. Leach, and A. S. Whittaker. 2009. Blast testing of ultra-high performance fibre and FRP-retrofitted concrete slabs. *Engineering Structures* 31:2060-2069.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE
July 2015

2. REPORT TYPE
Final report

3. DATES COVERED ***(From - To)***

4. TITLE AND SUBTITLE
State-of-Practice on the Dynamic Response of Structures Strengthened with Fiber Reinforced Polymers (FRPs)

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
Esteban Pabon, Yomar Otero, Felipe J. Acosta, Robert D. Moser, C. Kennan Crane, and Brian H. Green

5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

U.S. Army Engineer Research and Development Center
Geotechnical and Structures Laboratory
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER

ERDC/GSL SR-15-2

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Headquarters, U.S. Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT
Fiber Reinforced Polymers (FRPs) are one the most innovative materials for strengthening and retrofitting structures and structural elements due to their high modulus of elasticity, light weight, and other valuable properties. FRPs are often used in reinforced concrete, since concrete has a low modulus of elasticity and is more susceptible to exhibit cracks and brittle behavior. However, FRPs are not frequently used to repair steel structures because they are more ductile and have a relative high modulus of elasticity compared to concrete. In this work, a literature review for retrofitting structures using FRPs is presented. It is divided into two principal categories, concrete structures and steel structures. The review shows that there is a vast collection of information on strengthening and retrofitting concrete elements with FRPs. The results show that using FRPs in certain concrete elements provides increasing ductility and shear capacity of a structure leading to a favorable seismic behavior. There is also valuable information for strengthening concrete structures that are susceptible to blast loads. When compared to reinforced concrete, the applications and available research on strengthening steel elements with FRPs is limited. Specific FRP applications that are being developed are discussed in this report.

15. SUBJECT TERMS

Fiber reinforced polymer	Steel	Fiber
Retrofit	Concrete	Carbon
	Bridges	Glass

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER *(include area code)*
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED		68	

Standard Form 298 (Rev. 8-98)
Prescribed by ANSI Std. 239.18